WATERWORKS HANDBOOK

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COMPILED BY

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PREFACE

This book gives a usable compilation of information, old and new, for the waterworks engineer and superintendent, the designer, constructor, operator and inspector. The materials have been accumulated by the compilers in the course of their practice in various branches of waterworks engineering. The user is assumed to have some familiarity with mathematics, hydraulics, the natural sciences and waterworks construction, operation and maintenance, and to possess ordinary mathematical tables. For some cases, instead of rules or formulas, data have been given, from which the engineer can make his own determinations, exercising his judgment in accordance with local conditions and other data which he may possess. Mention of materials, apparatus, equipment, methods, formulas, persons or business concerns, does not necessarily imply approval by the compilers. Acknowledgment of large indebtedness to many persons is here made, especially to John H. Gregory, and Thomas C. Atwood, who read the proofs and made numerous helpful suggestions; credit to many others is given throughout the text.

For convenience of reference the book has been made partially self-indexing by grouping most of the contents under the following topics, naturally arranged: Sources of Supply; Collection and Works Therefor; Transportation by Aqueducts, Pipes, and Other Conduits; Distribution, Hydrostatics and Hydraulics; Materials Much Used in Waterworks; and Treatment by Filtration, Aeration, Chemicals and Other Means. Each topic is divided on natural and obvious lines and its divisions arranged sequentially. Material not readily classified under the above topics has been assembled under the heading of Miscellany on p. 635 *et seq.*

Specifications (*e.g.*, for water towers) have been stripped of obvious and usual non-technical matter and minor words and the substance packed into little space. The user will, of course, amplify and arrange in proper form when preparing a contract. Much information which will be useful in preparing specifications has not been so labelled, but has been put into natural places in the text.

THE AUTHORS

NEW YORK,
May, 1916

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WATERWORKS HANDBOOK

PART I
SOURCES OF WATER SUPPLY

CHAPTER I

RAINFALL OR PRECIPITATION

RAIN GAGES

Comparison of Rain Gages. Tests at Ithaca, N. Y., 1910, by N. Y. State Water Supply Commission, showed that variations in registration by different types of rain gages are not great. U. S. Weather Bureau gage was assumed as 1.000. (E. R., July 9, 1910.)

Make	Value on ground	Value 10 ft. above ground
Smithsonian	1 017	1 052
Fuertes	1 021	1 043
Dewitt conical .	0 984	0 966

Errors. Cleveland Abbe says that no error of more than 1 per cent. systematically attaches to gages of ordinary forms and of diameters between 4 and 44 in. Old rain gages, placed 8 ft. above ground, gave uniformly low results, on account of: (1) Loss by evaporation; (2) method of measuring snowfall; (3) hight above ground. Rain gages should preferably have no trees or buildings within 100 ft. and should be on open level ground, well away from the steep slope of the hillside. "A rain-gage, the mouth of which is 1 ft. above the level surface of the ground, will collect about 6 per cent. less water than if its top is at the level of the ground" (Conservation of Water by Storage, G. F. Swain).

Smithsonian 8-in. gage has a tin body, with a copper rim about 8 in. inside diameter, and a diaphragm soldered to sides of can, 5 in. below the top, to prevent evaporation. Rain passes through a $\frac{1}{12}$-in. hole in the center of diaphragm, which is dished $\frac{1}{2}$ in.; is poured out through a $\frac{1}{2}$-in. opening in the side of the gage just below the diaphragm, and is measured in a glass graduate about $3\frac{3}{4}$ in. diam. and $9\frac{3}{4}$ in. high, which gives a multiplication of 3.75. This gage, prior to 1874, was placed only slightly, if any, above the ground (in the vicinity of New York). The Smithsonian work was taken over by the Signal Service in 1870–74.

U. S. Weather Bureau Standard Gage, Fig. 1, consists of a galvanized-iron can, 8 in. in diam., and 20 in. high, provided with a funnel-shaped top, tightly fitting, which has a sharply beveled brass rim, accurately circular, so as to catch only the rainfall on a definite area. The bevel is on the outside, so that water striking it does not splash into the can. The funnel has an opening (made small to prevent evaporation) into an inner vertical brass cylinder of one-tenth the horizontal area of the rim of the funnel. The wetted length of a slender measuring rod graduated to tenths of an inch, inserted in this brass cylinder, measures 10 times the actual fall; and determines precipitation to hundredths. The displacement of the measuring stick is ignored, as it is small. The brass cylinder will hold 2 in. of rainfall, before it overflows into the annular space between the cylinders, called the overflow. To measure this overflow, pour into a convenient vessel, and then back into the cylinder.

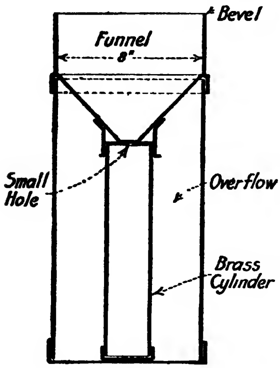


Fig. 1.

New England Water Works Assn. Rain Gage consists of a measuring cylinder, protected from puncture during handling by an outer cylinder with air space between. This annular air space is topped with a rim beveled outward to prevent spatter into the measuring cylinder. The principle of the apparatus lies in fact that area of the measuring cylinder (14.85 in. diam.) is such that 0.01 in. in the cylinder weighs 1 oz. Deducting tare from weight (any scales reading to ounces) gives rainfall in hundredths of inch.

Elaborate recording gages should be restricted to central stations, where they can be carefully watched; they need frequent attention. Severe winter weather is decidedly against delicate long-term apparatus.

Automatic Rain Gages are used only where they can be often inspected, as they are liable to derangement. Friez and Queen gages operate by the tipping, when filled, of a small bucket of known capacity; this causes a pen to move forward a definite distance on a revolving drum chart. Richard gage contains a tank with capacity of 0.04 in. of rain. After this is filled, electric contact is

Table 1. Automatic Rain Gages

Name	Maker	Cost	Capacity, in	Principle of operation	Diam of collecting funnel, in
Friez	J. P Friez, Baltimore, Md.	$118 75	-----	Tipping bucket	12 0
Marvin	J. P. Friez, Baltimore, Md	....	-----	Weighing	-----
FitzGerald	Builders Iron Fdy, Providence, R I.	$100 00	-----	Float gage	14.85
Queen	Queen-Gray Co, Philadelphia	$100.00	-----	Tipping bucket	-----
Richard:					
Model A.	Jules Richard, Paris	78 00	-----	Siphon	8 3
Model B.	Jules Richard, Paris	171 00	-----	Tipping bucket	8 3
Fergusson	International Instrument Co., Cambridge, Mass	80.00	6	Spring balance	8 0
Draper (new style).	Draper Mfg Co, N. Y.	75.00	5	Spring balance	8 0

Arranged from "American Sewerage Practice," Vol I, Metcalf and Eddy (McGraw-Hill Book Co., Inc, 1914), *q v.* for discussion of qualifications of the various instruments.

made, which starts apparatus operating a siphon; when tank is emptied, the recording pen moves back to zero. The Marvin type weighs the rain, deflection of the balance beam making electrical contact.* The FitzGerald gage operates on the float gage principle, the rising water operating a pen. Draper and Fergusson gages contain a can supported by a spring balance, the depression of which (by link motion) actuates a pen.

RAINFALL DATA

The "Water Year," in most regions, does not coincide with the calendar year; in some it is more appropriate to use the 12 months beginning Dec. 1, in others, beginning Aug., Sept. or Oct. 1. In northern latitudes A. F. Meyer† uses for rainfall the 12 months beginning Nov. 1, and for corresponding run-off the year beginning the following Mar. 1.

Precision.—Rainfall is customarily recorded to hundredths of inch, but this degree of precision is hardly warranted in most cases and the added labor gains nothing in accuracy, as shown by Table 2. Furthermore the greater refinement is not useful in practical engineering problems. Column A was computed using 2 decimal places; column B using only tenths.

Table 2. Comparison of Croton (N. Y.), Monthly Rainfall Totals from 1868 through 1910, in inches, using one and two decimals

	A	B		A	B
Jan.	180 99	180 8	July.	199.85	199 9
Feb.	178 02	177.8	Aug.	212.95	212 9
Mar	179.02	179.1	Sept.	180.22	179 9
Apr.	151 60	151.6	Oct.	168.74	168 9
May	165.93	166.0	Nov.	164.12	164 0
June	155 18	155 3	Dec.	169.15	169 4
			Total by adding above.	2105 77	2105.6
For complete Croton records, see p. 12.			Total, from summing yearly totals.	2105.8	2105.7

Estimating Precipitation. A German authority says all rain records should be increased 25 per cent. (probably an extreme view). Much faith should not be put in most American records previous to 1880. One method employed in estimating precipitation is to collect records from near-by stations and tabulate by monthly and yearly averages. From a station with a long record in the same or a comparable region, find mean annual rainfall for the year in question. Put this in one column; in the next column, compare this "long term mean" with the yearly mean, expressing the ratio in percentage. In case of only one year's record for any gage, a correction is made according to this ratio to get the long term mean.

In the absence of records for a desired area, the most logical guess is the mean of all adjacent records of same date. Adjacent localities sometimes

* Circular E, U. S. Weather Bureau.

† Trans. A. S. C. E., Vol 79, 1915, p. 1063.

produce widely varying records; *e.g.*, Albany and Troy, 7 mi. apart; Newark and New York, 10 mi. apart. Records for the same storm at stations but a few miles apart often vary. Greatest discrepancies occur in the months of thunderstorms.

Four methods of arriving at a mean annual rainfall: (1) Take the mean of all records, including broken ones. Averaging by months allows this, whereas a yearly average could be based only on complete annual records. (2) Combine the stations into groups equally distributed. Determine the mean for each group and take the average of these means as the average for the whole area. This has the advantage over No. 1 that too much weight is not given to any particular locality. (3) Take the mean of the inside stations, and the mean of the outside stations, of the area considered, averaging these two means. This method is good when there are a greater number of outside stations, as it gives more weight to the inside stations. (4) Isohyetal map.

Table 3. Average and Extreme Departures of Mean Rainfall by Short Periods from Mean Determined by Very Long Periods, Expressed in Per Cent.

Station	Long period, years	5 yrs.	10 yrs.	20 yrs.	40 yrs.
Padua	176	9 6	8 4	2 5	2 4
Klangenfust	88	9 5	8 1	2 6	2 6
Milan. . . .	100	7 0	5 9	2 7	2 0
Croton.	43	13 0	8 7	5 0	0 5
Sudbury	36	12 0	5 1	2 6	. .
†San Francisco	61	−26 2 +20 8	−16 4 +12 3	− 8 3 +10 8	− 2 2 + 7 0
Denver	39	20 0	−11 2 + 6 0	− 7 5 + 2 3	.
St Paul	74	−20 4 +21 6	−13 5 +20 3	− 8 3 + 9 8	− 1 4 + 2 9
New Orleans*	22 38	−16 8 +24 2	−15 0 +18 4	− 8 5 +14 1	4 0

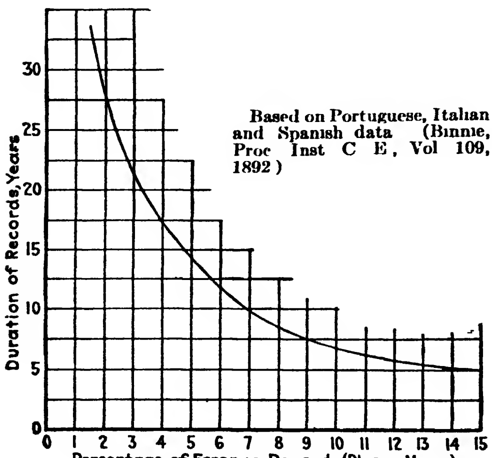


FIG. 2.—Probable error of short-time records.

Isohyetal Map. Procure a scale map of the locality to be studied. Locate the positions of all rain gages. At each gage mark its average rainfall. Averages should, if possible, be taken for same number of years and for same period. Treat the problem similarly to plotting contours on a topographical map, drawing "contours" through points of equal rainfall, using straight-line interpolations and extrapolations, where necessary. Good, long-

* Percentages for San Francisco, Denver, St Paul, and New Orleans give extreme variations, in excess (+) and deficiency (−), the others are averages. † Years 1839–1860, 1873–1910.

time records of large number of gages well distributed over the area should be available.

Rendering Records Comparable. Great differences in records exist; the following is a method of rendering them comparable suggested by Thaddeus Merriman: Any monthly rainfall that exceeds twice the monthly mean is excessive or unusual, and should be eliminated, as follows: For any month in which the rainfall exceeded twice the monthly mean, use the monthly mean, unless the rainfall for either the preceding or following month was less than half its monthly mean, in which case deduct only the excess of the 1 month over the deficiency of the 2 months. The value of the yearly rainfall so determined may be called the "mean annual dependable" rainfall.

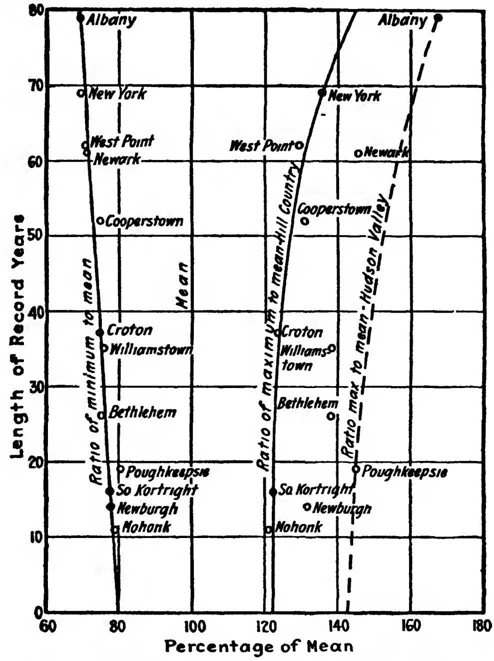


FIG. 3.—Relation of maximum and minimum to mean rainfall.

Where rainfall stations are widely separated, as in the Northwestern States, it is the experience of the U. S. Geological Survey that calculations are facilitated by expressing each year's rainfall for any station as a percentage of the mean annual of that station for many years. Percentages of several stations averaged together for any year convey an idea of the comparative dryness in the drainage basin, without introducing an error due to local conditions at the stations. Results thus obtained are said to be satisfactory, making averages for large areas much more accurate.

Table 4. Variation of Annual Rainfall by Decades; Northeastern U. S.

AVERAGE FOR EACH DECADE, INCHES

Decade	Lowell	Providence	New Bedford	Boston	Average
1832–1840	38 0	36 9	44 3	41 0	40 0
1841–1850	40 3	41 2	44 9	42.9	42 3
1851–1860	44 1	43 7	44 8	50 1	45 7
1861–1870	45 9	47 6	46 3	57.2	49 3
1871–1880	45 2	47 8	46 9	50 3	47 6
1881–1890	46 5	49 1	48 7	48 5	48 2
1891–1900	44 5	48 9	47.7	45 5	46 6
1901–1910	43.0	41 7	44 7	37.4	*41.7

From "Monthly Weather Review," U. S. W. B.

In comparing rainfalls at two localities of short records, both should be reduced to a common plane for comparison with long-time records. Data for Northeastern U. S. show that the proposition that the rainfall varies from the mean at the same rate for all stations similarly located is true 72 per cent. of the time. The smaller the area, the less is the error.

Table 5. Ratio of Total Rainfall in Any Seasonal Year (Oct. 1—Sept. 30) to Average Total Annual Rainfall for a Long Series of Years

Station	A	B	C	D	E	F	G	H	Mean
Minimum	0 67	0 70	0 70	0 71	0 69	0.75	0 74	0 73	0 71
Maximum	1 25	1 53	1.70	1 54	1 25	1 25	1.40	1 36	1.41
Average. . . .	1.00	1.02	0.99	1 01	0.99	1 00	1.06	0.99	1 01
Term of record.	1855 1905	1855 1905	1855 1905	1855 1905	1855 1905	1868 1905	1855 1905	1855 1905	1868 1905
No. of years.	50	50	50	45*	50	37	50	50	37
Mean rainfall.. (inches)	39 9	35.9	38.1	40 0	49.3	48 5	44 7	49 0	. . . . .

* Five scattering years are missing

Stations (in or near Hudson drainage basin) are Lansingburg, Troy and Albany, N. Y.; Williamstown, Mass , West Point, Croton watershed and New York City, N Y.; Newark, N. J., denoted by columns A to H, respectively. Observed total annual precipitation for each year at each station was divided by 37-yr mean, 1868–1905.

Averaging Rainfall on Watershed. Average monthly and yearly rainfalls on Catskill watersheds, New York City Water Works, are determined by a method of weights, each station being weighted in proportion by percentage that the area (generally a circle with center at the gage) which the gage is considered to govern, is to the area of total watershed. Weight × rainfall summed up and divided by 100 (weight of total water-shed) = average rainfall. This method gives results that agree well with the ordinary method of averaging when the rainfall over whole watershed has been fairly uniform. Differences of several tenths of an inch in the two methods are not uncommon, but are, as a rule, compensating. For heavy rainfall, the weighted method approaches more nearly the truth.

Factors Affecting Estimates of Rainfall. Usual pathway of storms across the region under consideration is an important element in determining the precipitation probable at a given point. Temperature is an important element affecting precipitation; elevation and barometric pressure have little effect, except as they affect temperature. One deg. F. drop for every 300 ft. increase in elevation, has proved a reliable basis for estimates. Owing to local conditions, such as adjacent buildings, Weather Bureau records in large cities are particularly unreliable. Meteorologists pretty thoroughly agree that the amount of precipitation on a given area is "directly proportional to its altitude and inversely proportional to its distance from the ocean or other large body of water over which the prevailing storm winds pass." (Kuichling.) Seaward slopes of mountain ranges which intercept moisture-laden winds receive copious precipitation, while landward slopes and interior regions receive less, often much less. (West coast of South America is an exception.) In some localities, precipitation on tops of mountains and high wooded hills is greater than in adjacent lowlands.

Snowfall Expressed in Inches of Rainfall. The ratio of snow to rain varies greatly with the character of the snowfall. U. S. Weather Bureau divides snowfall by 10 to get equivalent rain. Other authorities recommend division by 7. The one-tenth rule gives values 10 to 30 per cent. large for light snow at low temperatures. Careful measurements of falling snow at Potsdam, Prussia, 1896 to 1900, show that inches of snowfall should be divided by 12 to give equivalent rainfall. Studies also indicate a factor of 12 for Catskill Mountains, N. Y. Whenever practicable, the most accurate method is to melt without evaporation carefully collected portions of snow and measure like rain. Depth of snowfall should be measured promptly on an area unaffected by drifting, etc. Note whether light and dry or heavy and wet.

RAINFALL RECORDS

Explanation of Rainfall Table. The following tables were compiled mainly from U. S. Weather Bureau records, dropping the second decimal place, as explained on p. 3. Elevations refer to hight of gage above sea-level. "Totals" in the second column, and "Extent of Record" below each table,

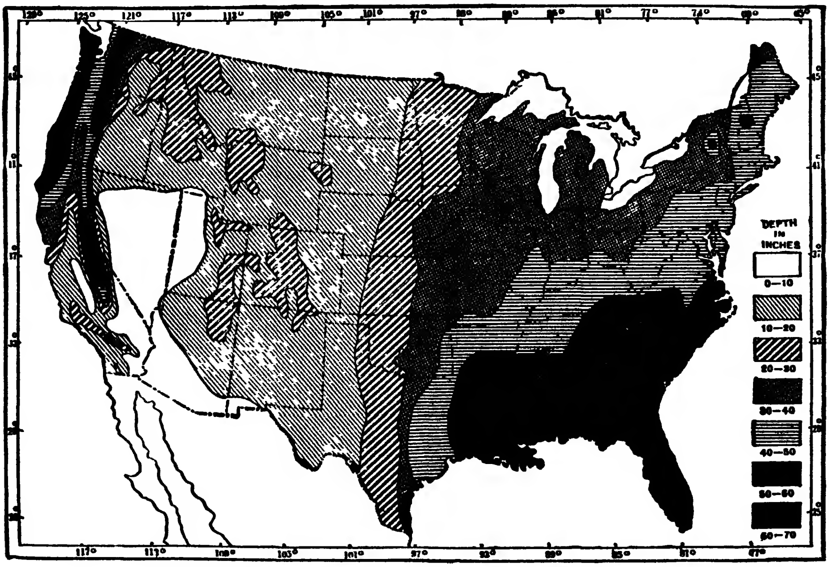


Fig. 4.—Mean annual rainfall of United States
From "Principles of Irrigation Engineering," Newell and Murphy, 1913.

enable an investigator in any year to bring the record quickly down to date by adding records of subsequent years. Records are inclusive of first and last dates given. Rainfall is expressed in inches, and includes snowfall expressed in inches of rain. Figures for the lowest 5 and highest 7 consecutive months are the total rainfalls during these periods; they are useful for studies in watershed development. Temperature is degrees Fahr. Places arranged geographically.

Table 6. Selected Long-term Rain and Temperature Records giving Monthly and Yearly Totals, Averages, Minima and Maxima, also Extreme Dry and Wet Seasons

(Minimum minimorum and maximum maximorum, also lowest and highest average, are underscored for each place)

Place	QUEBEC, Canada. Elev 296 ft						
	Rainfall, inches						Mean temp., ° F. (14 years) (1897-1910)
Time	Totals, 37 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	138.3	3 7	1875	0 5	1882	6 6	11
Feb	124 6	3 4	1878	1 0	1908	6 2	12
Mar.	127.2	3.4	1895	1 0	1877	6 2	24
Apr	81.0	2 2	{1883, 1886}	0 7	1880	6 6	38
May	116.6	3.2	1887	0 3	1874	6.9	51
June.	139 8	3.8	1881	1 3	1901	9 2	61
July.	154 4	4 2	1877	0 5	1900	7.1	67
Aug	143.7	3 9	1905	1 4	1890	7 6	63
Sept	137.3	3 7	{1874, 1903}	1 1	1889	8 8	56
Oct.	119 8	3 2	1895	0 9	1896	7 0	44
Nov.	125.3	3 4	1876	0 9	1878	7 1	30
Dec	129 6	3 5	1892	1 1	1878	6 8	17
Year	1536.2	41 5	1905	32 3	1879	52 4	40

Lowest 5 consec. months, Oct., 1892–Feb., 1893, 8.5.
Highest 7 consec. months, June–Dec., 1879, 36.1.
Extent of record 37 yrs., 1874–1910.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	TORONTO, Ontario. Elev. 350 ft. until 1908, 377 ft. since						
	Rainfall, inches						Mean temp., ° F. (70 yrs.)
Time	Totals, 37 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	103 4	2 8	1879	1 2	1886	5 5	22
Feb	91 3	2 4	1877	0 3	1900	5 0	22
Mar.	89 2	2 4	1905	0 5	1876	5 7	29
Apr.	82.5	2 2	1881	0 1	1909	5 4	41
May	103.0	2.8	1891	0 5	1894	9 1	53
June.	98 4	2.7	1899	0 6	1892	5 8	63
July.	107.8	2 9	{1887, 1898}	0 7	{1878, 1883}	5 6	68
Aug.	96.0	2.6	1876	0 0	1878	7 1	67
Sept	102.3	2 8	{1877, 1897, 1903}	0 4	1878	7 7	59
Oct.	90 0	2 4	1901	0 5	1898	5.4	47
Nov.	98 9	2 7	1904	0 1	1877	5 6	36
Dec.	93.6	2 5	1877	0 5	1889	4 9	26
Year	1156 7	31 3	1874	24 3	1878	48 5	44

Lowest 5 consec. months, Feb.–June, 1895, 6.0.
Highest 7 consec. months, June–Dec., 1878, 33.6.
Extent of record, 37 yrs., 1874–1910.
Lowest Jan. temperature, −7°.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	BOSTON, Mass. Elev. 125 ft.						
	Rainfall, inches						Mean temp., ° F.
Time	Totals, 93 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	357.3	3.8	1849	0 4	1836	8 8	27
Feb. ...	344.7	3.7	1901	0 7	1869	10.0	28
Mar.. .	390.4	4.2	{1834 1894}	1.0	1864	11 8	35
Apr. ...	365.0	3.9	1844	0 2	1857	10.8	45
May. .	341 0	3.7	1826	0 2	1868	10 4	57
June .	296 0	3 2	1853	0.3	1858	8 1	66
July. .	326.7	3.5	1849	0.8	1863	12 4	71
Aug .	386 3	4.2	1883	0.3	1826	12.1	69
Sept ...	325.6	3.5	1884	0 2	1868	12 0	63
Oct ...	344 1	3.7	1897	0 4	1890	8 8	52
Nov. .	383 2	4 1	1908	0.7	1840	11 6	41
Dec	355 1	3 8	1828	0 3	1869	9 0	32
Year .	4213 8	45 3	1822	27 2	1863	67 7	49

Lowest 5 consec. months, Oct., 1818–Feb., 1819, 7.9.
Highest 7 consec. months, Oct., 1869–Apr., 1870, 47.9.
Extent of record, 93 yrs., 1818–1910.
Lowest Jan. temperature, − 13° (36 yrs.).
In 47 days, Sept. and Oct., 1914, but 0.21 in. fell. (E.N., Apr. 8, 1915.)

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	SUDBURY WATERSHED, Mass Elev 200 ft.						
	Rainfall, inches						Mean temp., ° F (13 yrs)
Time	Totals, 36 yrs	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	150 2	4 2	{1876 1901}	1 8	1891	7 0	26
Feb......	152 8	4 2	1877	0 7	1900	9 1	25
Mar......	158.1	4 4	1910	0 8	1877	8 4	37
Apr	126 6	3 5	1892	0 8	1904	8 9	47
May.	120 2	3 3	1903	0 9	1901	7 2	57
June .	113.4	3.2	1908	0 9	1903	9 2	66
July.	128.2	3.6	{1885 1888}	1 4	1876	9 0	71
Aug	138 6	3.8	1883	0 7	1898	8 2	68
Sept	126 7	3.5	1877	0 3	1907	8 8	62
Oct.. .	141.4	3.9	1897	0 5	1895	10 7	51
Nov...	138.1	3.8	1908	1 0	1888	7.2	39
Dec . .	137.1	3.8	{1875 1877}	0.9	1901	9.7	28
Year .	1631.2	45.3	1883	32 8	1878	57.9	48

Lowest 5 consec. months, Apr.–Aug., 1899, 10.4.
Highest 7 consec. months, Sept., 1899–Mar., 1891, 41.7.
Extent of record, 36 yrs., 1875–1910.
Lowest Jan. temp., − 24° (13 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	WACHUSETT WATERSHED, Mass. Elev. 400 ft.						
	Rainfall, inches						Mean temp., ° F.
			Minimum		Maximum		
Time	Totals, 14 yrs.	Average	Year	Amount	Year	Amount	
Jan	53 2	3 8	1901	1 8	1898	6 6	24
Feb.	56.0	4 0	1901	1.1	1900	8 7	23
Mar.	59.5	4 2	1910	1 1	1899	6 8	34
Apr	55 7	4 0	1899	1.9	1901	9 6	45
May	48 1	3.4	1905	0.8	1901	7 0	57
June	58 2	4 2	1908	1.3	1903	10 4	65
July	59 0	4 2	1910	1.5	1897	8 6	71
Aug.	59 2	4 2	1907	1.3	1898	10 6	68
Sept	55 0	3 9	1908	1 0	1907	9 5	62
Oct.	46.7	3.3	1897	0 9	1898	7 2	51
Nov.	47.6	3 4	1902	0 9	1897	7 6	38
Dec.	60 8	4 3	1899	2 0	1901	9 4	27
Year	659 1	47 1	{1908 1910}	37 8	1898	57.9	47

Lowest 5 consec. months, Sept., 1908–Jan., 1909, 10.6.
Highest 7 consec. months, Aug., 1898–Mar., 1899, 39.8.
Extent of record, 14 yrs., 1897–1910.
Lowest Jan. temp., −16° (13 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	ESOPUS CREEK WATERSHED, Catskill Mts, N. Y. (6 to 11 stations). Elev. 600–1200						
	Rainfall, inches						Mean temp., ° F. (9 years)
			Minimum		Maximum		
Time	Totals, 9 yrs, 4 months	Average	Year	Amount	Year	Amount	
Jan	33 5	3 8	{1911 1912}	2 4	1910	7 4	27
Feb	30 9	3 4	1907	1 7	1909	6 9	23
Mar	35 7	4.0	1910	0 9	1913	7 7	35
Apr.	41 2	4.6	1907	2 2	1910	9 6	47
May	37 0	4.1	1911	1 1	1908	9 1	58
June.	32 6	3 6	1913	1 1	1906	6.2	66
July	29.5	3 3	1913	1 7	1908	6.3	72
Aug	36.4	4.0	1907	1 1	1912	7.3	69
Sept	45.3	4 5	1914	0 6	1907	11 2	62
Oct.	45.0	4 5	1910	1 1	1911	7 5	52
Nov.	33.7	3 4	1908	0 6	1907	6 9	40
Dec.	36 5	3 6	1910	2 2	1907	5 6	29
Year.	419 6	46 6	1914	39 7	1912	50 1	48

Lowest 5 consec. months, Oct., 1910–Feb., 1911, 11.1.
Highest 7 consec. months, Sept., 1907–Mar., 1908, 43.2.
Extent of record, 9 yrs., 4 months, Oct. 1, 1905–Dec., 1914, except 1905 total.
Mar., 1915, averaged 0.25 (0.08 to 0.66)—lowest rainfall in the 10 yrs. of Board of Water Supply records.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	PROVIDENCE, R. I. Elev. 160 ft.						
	Rainfall, inches						Mean temp., °F. (24 yrs.)
Time	Totals, 79 years	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	322.3	4.1	1843	0.6	1891	8.1	31
Feb	305.7	3.9	1877	0.3	1886	11.3	31
Mar.	327.7	4.1	1855	0.8	1876	9.8	36
Apr	295.8	3.7	1844	0.7	1904	9.4	44
May	289.2	3.7	1903	0.6	1868	10.6	53
June	257.2	3.3	1832	0.3	1842	9.7	62
July	258.6	3.3	1838, 1909	0.6	1898	10.3	68
Aug	318.3	4.0	1854	0.3	1875	8.8	69
Sept	267.5	3.4	1855	0.2	1888, 1899, 1907	9.2	64
Oct	287.8	3.6	1897	0.5	1890	9.2	55
Nov.	312.2	4.0	1890	0.7	1854	9.2	45
Dec	313.6	4.0	1875	1.0	1901	9.4	37
Year	3556.0	45.0	1846	30.5	1898	63.5	50

Lowest 5 consec. months, July–Nov., 1837, 7.2.
Highest 7 consec. months, Aug., 1888–Feb., 1889, 43.0.
Extent of record, 79 yrs., 1832–1910.
Lowest Jan. temp., – 8° (24 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	NEW BEDFORD, Mass. Elev. 88 ft						
	Rainfall, inches						Mean temp., °F (12 yrs.)
Time	Totals, 100 yrs	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan.	400.6	4.0	1839	0.8	1836	9.5	32
Feb	388.0	3.9	1818	0.9	1814	8.3	31
Mar	433.8	4.3	1853	1.3	1890	9.8	41
Apr	394.7	3.9	1846	1.2	1841	9.3	50
May.	396.4	4.0	1822	0.6	1868	9.4	61
June.	307.3	3.1	1912	0.1	1862	8.1	70
July	323.3	3.2	1909	0.7	1830	12.0	76
Aug	417.3	4.2	1854	0.2	1826	18.7	73
Sept	350.8	3.5	1865	0.3	1850	12.1	66
Oct	393.5	3.9	1874	0.6	1890, 1913	10.1	58
Nov.	424.5	4.2	1899	1.1	1897	9.7	46
Dec.	415.1	4.2	1828	0.4	1901	10.0	36
Year..	4645.4	46.4	1846	34.5	1829	65.4	52

Lowest 5 consec. months, Sept., 1837–Jan., 1838, 9.4.
Highest 7 consec. months, July, 1830–Jan., 1831, 47.5.
Extent of Record, 100 years, 1814–1913. Ref. X. H. Goodnough, E. N., Nov. 19, 1914.
Lowest Jan. temperature, – 7° (12 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	CROTON WATERSHED, N. Y. Elev. 200–600 ft.						
	Rainfall, inches						Mean temp., °F. (1912)
			Minimum		Maximum		
Time	Totals, 43 yrs	Average	Year	Amount	Year	Amount	
Jan	180.8	4.2	1896	1.1	1891	9 1	25
Feb ..	177.8	4.1	1901	0 8	1900	7.7	31
Mar. ..	179.1	4.2	1910	0 7	1877	8.1	40
Apr.... .	151.6	3 5	1892	1 1	1901	8 2	51
May ...	166.0	3.9	1887	0 3	1868	8.8	65
June .	155 3	3.6	1873	0 7	1903	11 3	72
July.	199 9	4 6	1868	2 1	1897	12 5	77
Aug ..	212.9	5 0	1899	0.6	1898	11.5	72
Sept.....	179.9	4.2	1881, 1884	0.8	1882	14 6	67
Oct . .	168.9	3 9	1879	0 7	1869	9 5	61
Nov. ..	164 0	3 8	1902	0 9	1889	8 5	47
Dec	169 4	3 9	1892	1 0	1901	8 8	39
Year	2105 7	49 0	1880	36 9	1901	63 7	54

Lowest 5 consec. months, Dec , 1871–Apr., 1872, 10.8.
Highest 7 consec. months, Mar.–Sept., 1901, 46.8.
Extent of record, 43 yrs., 1868–1910.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	ALBANY, N. Y. Elev. 85 ft.						
	Rainfall, inches						Mean temp., °F. (86 yrs)
			Minimum		Maximum		
Time	Totals, 85 years	Average	Year	Amount	Year	Amount	
Jan	225 0	2 7	1860	0 1	1836	7.3	23
Feb	209.4	2 5	1856, 1877	0 4	1870	5.2	24
Mar. .	229.8	2 7	1855, 1910	0.5	1843	7.4	33
Apr ..	230 5	2 7	1892	0 6	1857	7.0	47
May	294 3	3 5	1903	0 2	1833	8.5	59
June	341 4	4 0	1864	0 8	1862	8 7	68
July.....	352.3	4 1	1849	0 7	1871	9 4	72
Aug .	325 7	3 8	1854	0 6	1871	10 6	70
Sept . .	283 2	3 3	1908	0 6	1890	8 9	62
Oct.... .	287.2	3.4	1882	0.3	1869	13 5	50
Nov. ..	252 9	3 0	1908	0 4	1830	7 3	39
Dec ..	223 2	2 6	1828	0 2	1878	6 2	28
Year ..	3254 9	38 3	1905	27 0	1871	56 8	48

Lowest 5 consec. months, Jan.–May, 1860, 5.1.
Highest 7 consec. months, Feb.–Aug., 1871, 45.3.
Extent of record, 85 yrs., 1826–1910.
Lowest Jan. temp., − 34° (36 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	PEQUANNOCK WATERSHED, Newark, N. J. Elev. 700 ft.						
	Rainfall, inches						Mean temp., °F. (20 yrs)
Time	Totals, 19 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	71 1	3 7	1896	1 8	1905	6.9	27
Feb......	80.1	4.2	1895	0 2	1896	9.3	26
Mar......	75 3	4.0	1910	0 9	1901	7.3	37
Apr......	79 5	4.2	1896	0 8	1909	8.8	49
May.....	76 0	4.0	1903	0.3	1908	7.8	58
June.....	82 5	4.3	1898	2.1	1903	11 0	65
July......	90.6	4.8	1894	1.0	1897	14.2	70
Aug......	90.1	4.7	1900	2.0	1901	12 1	67
Sept......	88 2	4.6	1895	1.0	1907	12 6	62
Oct......	83.5	4.4	1909	1.0	1903	13.7	51
Nov......	61.4	3 2	1908	0.8	1907	6.4	40
Dec	74 3	3.9	1896	0.9	1901	9 0	30
Year .	952 6	50.1	1895	36 7	1903	64 8	49

Lowest 5 consec. months, Oct., 1900–Feb., 1901, 11.5.
Highest 7 consec. months, Mar., 1901–Sept., 1901; Sept., 1907–Mar., 1908, 49.3.
Extent of record, 19 yrs., 1893–1911.
Lowest Jan. temp., – 26°.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	PHILADELPHIA, Pa. Elev 117 ft						
	Rainfall, inches						Mean temp, °F. (33 yrs)
Time	Totals, 91 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	298.5	3.3	1821	0.5	1841	7.8	32
Feb... .	287.3	3.2	1864	0 6	1896	6.9	33
Mar......	311.6	3.4	1910	0.4	1859	7.0	40
Apr.....	302.0	3.3	{1847, 1881}	0.6	1874	9.8	51
May.....	337.7	3.7	1826	0 2	1894	9.5	62
June.....	337.9	3 7	1885	0.7	1867	11.0	71
July......	368.2	4.0	1894	0.8	1842	11 8	76
Aug......	403.5	4 4	1821	0.4	1867	15.8	74
Sept.....	322.8	3.5	{1846, 1884}	0.2	1882	12.1	67
Oct......	295.7	3.2	1892	0.3	1833	10.0	56
Nov......	303.6	3.3	{1890, 1908}	0.8	1846	8.0	45
Dec	308 0	3.4	1828	0 3	1823	7.4	36
Year.	3876 6	42.6	1825	29.7	1867	61.2	54

Lowest 5 consec. months, July–Nov , 1881, 8.1.
Highest 7 consec. months, Feb.–Aug., 1867, 47.7.
Extent of record, 91 yrs., 1820–1910.
Lowest Jan. temp., – 5° (38 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	BALTIMORE, Md. Elev. 123 ft.					
	Rainfall, inches					Mean temp., ° F. (33 yrs.)
Time	Totals, 94 yrs.	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan.. ...	266.9	2.8	1825, 1862	0.6	1859	7.1	33
Feb......	290.3	3.1	1864	0 1	1820	8.2	35
Mar......	344.0	3.7	1910	0.5	1829	9.1	42
Apr......	296.4	3.2	1847, 1855	0.4	1839	9.1	53
May.....	324.6	3.5	1826, 1866	0.2	1858	9.1	64
June.....	332.7	3.5	1853	0.6	1836	9.2	73
July......	378.5	4.0	1864, 1870	0.4	1889	11.0	77
Aug......	359.2	3.8	1821, 1844	0.3	1817	10.0	75
Sept.....	324.9	3.5	1884	0 1	1821	10.7	69
Oct.	283 1	3.0	1874	0 2	1833	7 9	58
Nov.... .	282 1	3 0	1870	0 3	1852	7 9	46
Dec	306 0	3 3	1828	0 3	1839	8 8	37
Year...	3789 6	40.3	1870	22 4	1889	62 4	55

Lowest 5 consec. months, Feb.–June, 1856, 6.6.
Highest 7 consec. months, Mar.–Sept., 1889, 44.4.
Extent of record, 94 yrs., 1817–1910.
Lowest Jan. temp., – 6° (39 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	WASHINGTON, D C. Elev. 112 ft.								
	Rainfall, inches								Mean temp., ° F. (33 yrs.) to 1908
Time	Totals	Average	No. in averages	Missing years	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan	222 8	3.1	71	1830–38, 40, 43–45, 49–51	1872	0 2	1882	7 1	33
Feb.	221 5	3.1	72	1830–38, 43–45, 49–51	1864	0.3	1884	6 8	34
Mar .	248 1	3.4	72	1830–38, 43–45, 49–51	1910	0.6	1867	9 2	42
Apr. .	236 9	3.3	72	1830–38, 43–45, 49–51	1847	0.3	1889	9 1	53
May. .	264 0	3.7	72	1830–38, 43–45, 49–51	1826	0.8	1889	10.7	64
June...	270.0	3.8	72	1830–38, 43–45, 49–51	1864	0 8	1900	10 9	73
July...	305 0	4.3	71	1827, 30–37, 42–45, 49–51	1872	0.8	1886	10.6	77
Aug...	284.1	4 0	71	1827, 30–37, 42–45, 49–51	1854	0.6	1906	14 4	74
Sept...	230.6	3.2	71	1827, 30–37, 42–45, 49–51	1884	0.1	1876	10 8	68
Oct....	224.7	3.1	72	1830–37, 42–45, 49–51	1874, 1892, 1895	0.3	1866	10.6	57
Nov...	184.5	3.6	72	1830–37, 42–45, 49–51	1828	0 0	1877	7.2	45
Dec. ..	215 9	3.0	72	1830–37, 42–45, 49–51	1889	0.2	1901	7.6	36
Year.	2893 2	40 7	71	1827, 30–35, 38, 40, 42–45, 49–51	1826	18.8	1878	62.1	55

Lowest 5 consec. months, Aug.–Dec., 1828, 4.2.
Highest 7 consec. months., Jan.–July, 1886, 46.4.
Extent of record, 1824–1910.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	ATLANTA, Ga. Elev. 1174 ft.						
	Rainfall, inches						Mean temp., ° F. (44 yrs.)
Time	Totals, 47 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	215.2	4.7	1876	1.2	1883	15.8	42
Feb......	234.4	5.1	{1898, 1906}	0.6	1873	12.0	45
Mar......	256.7	5.6	1905	0.9	1880	11.9	52
Apr......	184 2	4.1	1896	0.6	1874	10.4	61
May.....	154.2	3.4	1897	0.3	1871	7.8	69
June.. ..	180 3	4.0	1868	0.5	1884	10.7	75
July......	190 7	4 2	1881	0.6	1887	14.1	78
Aug.. .	201.5	4.5	1877	1.0	1874	10.0	76
Sept... .	153.1	3.4	{1884, 1897}	0.1	1888	14.3	72
Oct......	103.5	2.3	{1886, 1891}	0.0	1868	8.9	62
Nov....	153.1	3.3	1890	0 2	1880	8.2	52
Dec...	214 3	4.6	1889	0.6	1859	11.2	45
Year..	2213.1	49.2	1904	33.1	1888	65.0	61

Lowest 5 consec. months, July–Nov., 1884, 8.1.
Highest 7 consec. months, Sept., 1880–Mar., 1881, 52.7.
Extent of record, 47 yrs., 1859, 1865–1910. Missing: Jan.–Oct., 1865; Apr.–Nov., 1867, and corresponding totals.
Lowest Jan. temp., – 2° (34 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	PITTSBURGH, Pa. Elev. 842 ft.						
	Rainfall, inches						Mean temp., ° F. (33 yrs)
Time	Totals, 71 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan.....	181.1	2.6	{1851, 1859}	0.4	1888	6.2	31
Feb.....	170.5	2.5	1841	0.1	1887	6.5	32
Mar......	205.5	3.0	1910	0.4	1898	5.4	39
Apr......	204 5	3.0	1849	0.8	1852	9.3	51
May.....	229.8	3.4	{1845, 1879, 1880}	1.2	1858	6.6	63
June.....	248.0	3.7	1894	0.6	1855	7.6	71
July......	260.7	3.9	{1894, 1909}	1.2	1887	9.5	75
Aug......	216.3	3.2	1894	0.4	1864	8.3	72
Sept.....	194 1	2.8	1908	0.7	1864	8.2	66
Oct......	173.7	2.6	{1874, 1897}	0.1	1873	6.2	55
Nov......	172.7	2.5	1904	0.2	{1855, 1897}	5.1	43
Dec.....	200.0	2.9	1861	0.4	1890	5.6	35
Year	2324.4	35.8	1839	25.3	1890	50.6	53

Lowest 5 consec. months, Aug.–Dec., 1908, 6.8.
Highest 7 consec. months, Mar.–Sept., 1865, 38.3.
Extent of record, 71 yrs., 1836–1867, 1872–1910. Missing: Jan.–July, 1836; Apr.–Dec., 1838; Jan.–Aug., 1866; May–Dec., 1867, and corresponding totals.
Lowest Jan. temp., – 12° (34 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	CHICAGO, Ill. Elev. 823 ft.						
	Rainfall, inches						Mean temp., ° F. (36 yrs.)
			Minimum		Maximum		
Time	Totals, 40 yrs.	Average	Year	Amount	Year	Amount	
Jan.	84.4	2 1	1879	0.5	1897	4.5	24
Feb	91.1	2.3	1877	0.1	1881	6.0	25
Mar.	100 5	2.5	1910	0.3	1877	5 4	35
Apr	116 2	2.9	1899	0 1	1909	7.7	46
May	144 5	3.6	1897	0.8	1883	7.3	57
June	139.5	3.5	1904	0.6	1892	10.6	66
July.	141.4	3.5	1874, 1894	0.6	1889	9.6	72
Aug.	122.7	3.1	1893	0 2	1885	11.3	71
Sept.	123 5	3 1	1891	0 3	1894	8 3	65
Oct	95 6	2 4	1897	0 2	1883	7.4	53
Nov.	100.6	2.5	1903, 1904	0.3	1877	6.1	39
Dec.	83.6	2.1	1896	0 2	1895	6.8	29
Year	1341 4	33.5	1901	24 5	1883	45.9	49

Lowest 5 consec. months, Dec., 1898–Apr., 1899, 5.5.
Highest 7 consec. months, May–Nov. 1883, 33.7.
Extent of record, 40 yrs., 1871–1910.
Lowest Jan. temp., – 20° (38 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	DETROIT, Mich. Elev. 730 ft						
	Rainfall, inches						Mean temp., ° F. (33 yrs.)
			Minimum		Maximum		
Time	Totals, 40 yrs.	Average	Year	Amount	Year	Amount	
Jan	80 7	2 0	1902	0.6	1874	5.0	24
Feb.	92.0	2.3	1877	0.0	1881	6 4	25
Mar.	93 9	2.3	1910	0.4	1876	5 5	33
Apr.	92 9	2.3	1899	0.5	1880	6 2	45
May	132 1	3.3	1877	0.9	1892	7.8	58
June	152.2	3.8	1895	0.6	1892	8.3	68
July.	140.0	3.5	1877	0.0	1878	8.8	72
Aug	109.0	2.7	1889, 1894	0.2	1877	7.3	69
Sept.	102.3	2.6	1877	0.4	1902	6.5	63
Oct	92.4	2.3	1892	0.3	1881	6.5	52
Nov	101.2	2.5	1904	0.2	1891	5.3	39
Dec.	96.0	2.4	1900	0.4	1878	4.8	29
Year	1283.9	32.1	1889	21.1	1880	47.7	43

Lowest 5 consec. months, Sept., 1874–Jan., 1875, 5.4.
Highest 7 consec. months, Apr.–Oct., 1880, 36.5.
Extent of record, 40 yrs., 1871–1910.
Lowest Jan. temp., – 16° (36 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	DULUTH, Minn. Elev 1133 ft						
	Rainfall, inches						Mean temp., ° F. (38 yrs)
Time	Totals, 40 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	39.6	1 0	1904	0 2	1886	2 3	10
Feb. ...	40.1	1 0	1877	0 1	1884	2 7	13
Mar.... .	59.7	1.5	1883 1910	0 4	1894	4.3	24
Apr... .	82.5	2.1	1873	0 3	1894	5 8	39
May ..	133.4	3.3	1900	0 6	1879	8 0	48
June. ...	172 5	4.3	1910	0 1	1874	10 9	58
July. . .	155 6	3 9	1875	0 5	1909	10 8	66
Aug. .	138 4	3 5	1878	0 5	1889	7.8	65
Sept.....	150 1	3.8	1892	0 3	1881	11 5	57
Oct.. ..	104.2	2.6	1895	0 1	1894	5.0	45
Nov.	60 7	1 5	1904	0 2	1896	3 4	29
Dec.. .	48.9	1 2	1905	0 1	1879	3 9	17
Year	1184 2	29 5	1910	18 1	1879	45 3	39

Lowest 5 consec. months, Jan.–May, 1900, 2.9.
Highest 7 consec. months, May–Nov., 1879, 36.4.
Extent of record, 40 yrs., 1871–1910.
Lowest Jan. temp., – 41° (34 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	ST. PAUL, Minn. Elev. 837 ft						
	Rainfall, inches						Mean temp., ° F (38 yrs)
Time	Totals, 75 yrs	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	66 7	0.9	*	0 0	1881	4 3	12
Feb	59 2	0 8	†	0 0	1869	2 8	15
Mar. .	104.4	1.4	1853	0 0	1849	4.1	28
Apr	176.6	2.4	1848	0 2	1860 1862	5 8	46
May	253 2	3 4	1900	0 3	1906	10 4	58
June.	300.5	4 1	1863	0 0	1874	11 7	67
July.	262.9	3 5	1894	0 1	1838	11 1	72
Aug.	256 3	3 4	1894	0 4	1849	9 6	70
Sept.	253.5	3.4	1882	0 3	1869	10 6	61
Oct....	156.7	2.1	1853 1857	0 0	1900	7 6	48
Nov.	104.0	1 4	1848 1904	0.1	1857	5 8	31
Dec ...	72 9	1 0	1850	0 0	1857	3 0	19
Year	2042 8	27 6	1910	10 2	1849	49 7	44

Lowest 5 consec. months, Nov., 1852–Mar., 1853, 0.4. * 1853, 92, 98.
Highest 7 consec. months, Apr.–Oct., 1849, 40.7. † 1846, 53, 54, 64, 77.
Extent of record, 75 yrs., 1834–1910. Missing: Jan.–June, and total, 1836.
Lowest Jan. temp., – 41° (37 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	NEW ORLEANS, La. Elev. 51 ft.						
	Rainfall, inches						Mean temp., ° F. (39 yrs.) (1862-1900)
Time	Totals, 65 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan.. ...	289.0	4.5	1840	0 1	1881	11 2	54
Feb......	273.9	4.3	1892	0.0	1875	13.8	57
Mar......	292.7	4.6	{1860, 1898}	0.8	1903	14.6	63
Apr......	291.2	4.5	1891	0 3	1883	14 2	69
May.....	264 4	4.1	1898	0.0	1873	18.7	75
June.....	354 1	5.4	1907	1 0	1843	14 6	81
July......	415.1	6.4	1859	0.9	1869	15.5	82
Aug......	367.0	5.6	1884	0.9	1888	22 7	82
Sept.....	282 9	4 4	1839	0.1	1898	13 9	79
Oct.....	210 4	3 2	1874	0 0	1869	13 4	70
Nov... .	246 8	3 8	1903	0 2	1851	8 3	62
Dec	294 5	4 5	1889	0 7	1905	14 4	55
Year	3543 8	55 4	1899	31 1	1875	85 7	69

Lowest 5 consec. months, Oct., 1889–Feb., 1890, 6.1.
Highest 7 consec. months, Feb.–Aug., 1888, 63.2.
Extent of record, 65 yrs.: 1836, 1839–1860, 1868, 1870–1910, except Jan.–Mar., and total, 1868.
Lowest Jan. temp., + 15° (39 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	DENVER, Col. Elev. 5291 ft						
	Rainfall, inches						Mean temp., ° F. (31 yrs.) (1873-1903)
Time	Totals, 39 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan. ...	18 5	0 5	*	0.1	1883	2.4	29
Feb .	19 3	0 5	†	0 1	1909	1 4	32
Mar.	39 6	1.0	1908	0.1	{1891, 1905}	3.1	39
Apr.	81 4	2 1	1878	0 1	1900	8 2	48
May .	99.2	2 5	1886	0.1	1876	8.6	57
June	56 0	1.5	1890	0 0	1882	5.0	67
July.	68 5	1.8	1901	0 0	1895	4 3	72
Aug.	54 3	1.4	1900	0 1	1908	3.2	71
Sept.	38.2	1.0	{1879, 1892}	0 0	1909	3.8	63
Oct.	36.3	0.9	1876	0 1	1892	3.9	51
Nov.	22 2	0.6	{1899, 1901}	0 0	1886	1.9	39
Dec .	23 6	0 6	‡	0 0	1883	2.3	33
Year	556 4	14 3	1893	8 5	1909	23 0	51

Lowest 5 consec. months, Sept., 1879–Jan., 1880, 1.1.
Highest 7 consec. months, Mar.–Sept., 1909, 19.1.
Extent of record, 39 yrs., 1872–1910.
Lowest Jan. temp., – 29° (33 yrs.).
* 1873, 78, 88, 93, 1900, 1, 3, 4.
† 1876, 1901, 6, 8.
‡ 1881, 90, 95, 1905, 6.

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	FORT DAVIS, Texas. Elev. 4927 ft						
	Rainfall, inches						Mean temp., ° F. (18 yrs)
Time	Totals, 31–35 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan ..	17 8	0 5	A	0 0	1858	3 0	44
Feb .	16 6	0.5	A	0 0	1877	3.5	48
Mar. . .	14 0	0.4	A	0.0	1905	2.6	54
Apr. .	17 4	0.5	A	0 0	1855	3 6	62
May. .	34.5	1.0	A	0 0	1881	6 3	69
June	66 3	2.0	A	0 0	1873	6 8	74
July....	108 2	3.2	1903	0.2	1875	15 4	75
Aug. .	122 5	3.6	1871	0.2	1876	10 4	73
Sept ..	100.4	3.0	1869, 1871	0 1	1887	7.1	68
Oct.	45 7	1 3	A	0 0	1879	6 2	61
Nov.	18 9	0 6	A	0 0	1905	3 2	50
Dec.	19.5	0 6	A	0.0	1874	4.0	45
Year	558 1	18 0	1871	6.8	1875, 1876	27.7	60

Lowest 5 consec. months, 0.0. A. Several years.
Highest 7 consec. months, Mar.–Sept., 1875, 25.6.
Extent of record, 1855–1907, 53 yrs., omitting 1861–68; Jan.–May, and total, 1869; July–Dec., and total, 1891; 1892–1901; Apr., May, June, Nov., Dec., and total, 1906; Jan., Apr., Nov., Dec., and total, 1907.
Lowest Jan. temp., − 3° (16 yrs.).

Selected Long-term Rain and Temperature Records—*Cont'd*

Place	PHOENIX, Ariz. Elev. 1108 ft.						
	Rainfall, inches						Mean temp , ° F. (1896–1903)
Time	Totals, 35 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan	26 7	0 8	A	0 0	1897	3 7	52
Feb.	29.9	0.9	A	0.0	1905	4.6	56
Mar.	20 6	0 6	A	0 0	1905	2 4	60
Apr	12 1	0 3	A	0 0	1905	2 6	67
May	3 5	0 1	A	0.0	1893	1.0	75
June	2.6	0.1	A	0.0	1899	0.8	85
July.	31 7	1 0	A	0.0	1896	4.3	90
Aug	31 1	0.9	A	0.0	1881	2 2	89
Sept. .	22 6	0.7	A	0 0	1897	3.7	83
Oct .	11 8	0.4	A	0 0	1907	2.0	71
Nov.. ..	23.1	0.7	A	0.0	1905	3.6	61
Dec .	29.2	0.9	A	0.0	1883, 1889	3.4	52
Year..	237.2	7.4	1885	3.8	1905	19 7	70

Lowest 5 consec. months, 0.0. A. Several years.
Highest 7 consec. months, Dec., 1904–June, 1905, 13.6.
Extent of record, 35 yrs., 1876–1910. Missing: Jan., June–Aug., and total, 1876; July–Dec., and total, 1887; Jan.–Mar., Oct., Dec., and total, 1888.
Lowest Jan. temp., + 12° (17 yrs.).

In southern California it is necessary to store enough water to provide for the consumption of from 5 to 7 successive dry years. Run-off generally takes place from Jan. to May, and the stream beds are dry for the remainder of the year. (M. M. O'Shaughnessy, T. A. S. C. E., Vol. 75, p. 29, 1912.)

Table 8. Temperature and Rainfall in Desert Region, Southwest U. S.
(NAT'L. GEOGRAPHIC MAG., AUG., 1910)

Place	Fort Yuma, Ariz.			Phoenix, Ariz.			Tucson, Ariz.			Mohave, Cal.		
Extent of record, years	26	26	20	17	17	22	6	6	15	5	5	26
Month	Max temp	Min temp	Mean rain-fall	Max temp.	Min temp	Mean rain-fall	Max temp	Min temp.	Mean rain-fall	Max temp	Min temp	Mean rain-fall
Jan	81	22	0.42	87	12	0.80	80	17	0.79	70	16	0.95
Feb.	92	25	0.61	92	19	0.70	83	17	0 90	78	20	0.92
Mar	100	31	0 26	97	24	0.58	92	22	0 77	83	26	0.75
Apr.	107	38	0 07	105	30	0.30	30	95	0.28	100	35	0 17
May	112	44	0 04	113	35	0 13	102	32	0 14	102	38	0.03
June	117	52	Trace	119	33	0 10	112	48	0.26	107	48	0.05
July	118	61	0 14	116	46	1.03	108	59	2.40	115	64	0.08
Aug	115	60	0 35	116	49	0.88	109	57	2.60	112	57	0.04
Sept	113	50	0 15	114	39	0.64	107	49	1 16	104	45	0.07
Oct	108	41	0 28	105	34	0 37	98	29	0 64	93	40	0.25
Nov.	92	31	0 29	97	24	0 54	90	21	0 81	84	27	0.40
Dec	83	24	0 46	95	18	0 86	83	10	1 00	70	15	1 26
Year	118	22	2.84*	119	12	6.93	112	10	11 74	115	15	4 97

Two decimal places used on account of small quantities. Temperature is degrees Fahr. Rainfall in inches. * 26 yrs' observation.

Maxima Rates of Rainfall. (T. A. S. C. E., Vol. 54, 1905.) Record for July 21 and 22, 1894, at Boston, showed a downpour amounting to 1.08 in. in 22 min., when corrected for elevation of gage, or a rate of 2.94 in. per hr.; and this rate was maintained with almost absolute uniformity for the entire 22 min. The rain of Aug. 22, 1899, showed a precipitation of 2.05 in. in 60 min., also unusual intensities of 1.65 in. per hr. for 85 min., and 4.62 in. per hr. for 22 min. But one other storm having a greater intensity than 2 in. per hr. for 60 min. has been registered by a recording gage in eastern U. S., that of Aug. 3, 1898, at Philadelphia, reported by A. J. Henry, U. S. Weather Bureau, in *Journal* Western Soc. Engrs., Apr., 1899. It had intensities as follows: 7.2 in. per hr. for 5 min.; 5.1 in. per hr. for 15 min.; 4 in. per hr. for 40 min.; 3.8 in. per hr. for 70 min.; 3 in. per hr. for 110 min., Fig. 5. On July 1, 1915 at Boston all records for rain for 2-hr. and 5-hr. periods were broken: 4 to 6 A. M., 2.29 in.; 3 to 7 A. M., 3.37 in.; 4.5 in. for day.* This falls within Sherman's "maximum," Fig. 6.

Relation between intensity and duration is shown on Fig. 6. The upper Boston curve represents the maximum intensity for any period, so far as it may be determined from records. Its equation is $i = 38.64 \div t^{0.687}$, in which i = intensity of precipitation in in. per hr.; t = duration in minutes. No Boston records fall beyond this curve and but one Philadelphia point, that of Aug. 3, 1898. Lower Boston curve represents the greatest inten-

* E. N., Jul. 15, 1915.

Table 9. Temperatures and Rainfall in California, 1883–1908

U. S. Weather Bureau, San Francisco

Year	Red Bluff				Sacramento				Fresno				Los Angeles*				San Francisco*			
	Temperature			Rain-fall	Temperature			Rain-fall	Temperature			Rain-fall	Temperature			Rain-fall	Temperature			Rain-fall
	Mean	Max	Min.		Mean	Max	Min		Mean	Max.	Min.		Mean	Max	Min.		Mean	Max.	Min	
1883	61.5	107	19	13.8	59.9	104	22	13.5	—	—	—	—	61.6	104	28	14.1	55.6	95	36	15.4
1884	60.8	107	22	28.1	59.8	100	21	34.9	—	—	—	—	60.8	102	34	40.2	56.7	83	35	38.8
1885	64.4	108	33	29.6	62.5	105	34	20.7	—	—	—	—	63.0	108	36	10.5	57.8	87	43	24.9
1886	63.2	109	30	17.2	60.3	105	27	18.2	—	—	—	—	61.1	98	32	16.7	57.3	94	41	20.0
1887	64.4	112	27	13.6	60.3	100	28	13.3	—	—	—	—	61.7	100	34	16.0	56.5	97	33	19.0
1888	64.5	109	18	24.9	61.4	108	19	18.5	65.4	111	20	8.8	60.7	99	31	20.8	57.3	93	29	23.0
1889	63.3	111	26	32.9	60.9	104	31	27.5	64.3	112	27	12.3	63.0	103	32	33.3	57.9	89	39	36.9
1890	61.5	110	22	25.6	59.4	102	27	21.0	62.6	111	24	8.4	63.6	105	30	12.7	53.3	86	36	25.4
1891	62.4	114	26	23.0	60.6	106	26	15.6	63.0	114	26	8.9	63.0	109	33	12.8	56.6	100	37	21.1
1892	62.2	108	28	33.5	60.2	106	26	23.6	63.1	112	27	8.8	61.6	99	33	18.7	56.0	92	38	22.1
1893	60.6	106	27	24.4	58.8	103	28	16.6	61.0	109	28	9.4	61.5	92	31	22.0	54.3	90	36	17.9
1894	62.0	110	27	27.0	60.3	108	26	22.6	62.6	109	25	12.5	60.3	99	32	7.5	55.1	94	36	24.3
1895	62.2	108	28	22.6	60.2	102	28	17.4	62.5	110	26	10.4	61.7	100	34	12.6	55.6	89	38	17.1
1896	62.5	109	26	28.5	60.7	104	28	25.1	63.7	111	28	11.0	63.1	103	35	11.8	55.9	91	33	28.3
1897	62.0	109	27	20.1	59.8	105	24	15.3	62.3	110	23	8.4	61.8	97	30	14.3	55.0	92	38	16.4
1898	62.5	112	24	12.9	59.5	110	26	10.0	63.1	114	24	5.0	62.5	99	31	4.8	55.6	89	36	9.3
1899	62.4	109	26	28.1	59.6	102	30	21.1	62.6	111	24	10.5	62.0	100	33	8.7	55.0	94	34	23.2
1900	62.4	109	29	21.8	59.9	102	30	17.9	62.8	109	26	11.1	62.5	96	37	11.3	56.2	92	40	15.3
1901	63.1	111	25	25.5	60.1	105	26	18.5	63.5	110	27	8.1	62.1	97	31	12.0	55.2	91	37	19.8
1902	61.4	115	28*	35.5	59.2	107	29	17.9	62.2	110	24	7.9	61.2	94	32	13.1	55.4	83	38	19.2
1903	61.5	108	27	22.9	59.4	102	29	14.7	62.2	108	25	6.2	62.2	97	32	14.8	55.3	96	37	18.3
1904	62.8	108	29	34.0	60.1	102	32	21.0	63.7	109	28	13.2	63.9	97	35	11.9	56.4	101	38	24.7
1905	62.9	113	25	23.4	59.7	110	28	15.0	63.0	115	26	7.3	62.6	101	33	19.2	56.3	98	39	16.2
1906	63.0	110	30	40.4	60.4	104	30	30.7	63.2	111	27	16.0	62.9	105	34	21.5	56.2	85	40	26.3
1907	61.2	101	27	24.7	59.6	99	31	20.1	62.5	103	30	9.0	62.6	103	35	15.3	56.4	90	36	22.5
1908	62.3	114	25	17.9	59.7	103	28	11.2	63.0	114	29	7.0	61.9	96	37	13.7	55.0	89	35	16.4

Note. Red Bluff represents northern extremity of Sacramento Valley; Sacramento, central of that valley; San Francisco, central coast region; Fresno, central of San Joaquin Valley; Los Angeles, Southern California coast. Temperature, degrees Fahr.; rainfall, inches. * See also page 20.

sity for any period ordinarily necessary to consider in engineering design, storms of greater intensity being of rare occurrence. Its equation is $i = 25.12 \div t^{0.687}$. E. S. Dorr's: $i = 150 \div (t + 30)$, for periods greater than 20 min., differs but slightly from this curve. A. N. Talbot has noted that maxima rates of rainfall seem to be very uniform in different parts of the U. S. (east of the Rocky Mountains), and has deduced a maximum and an ordinary curve applicable to the whole country: "Maximum" curve (eastern U. S.), $i = 360 \div (t + 30)$; "Ordinary" curve, $i = 105 \div (t + 15)$.

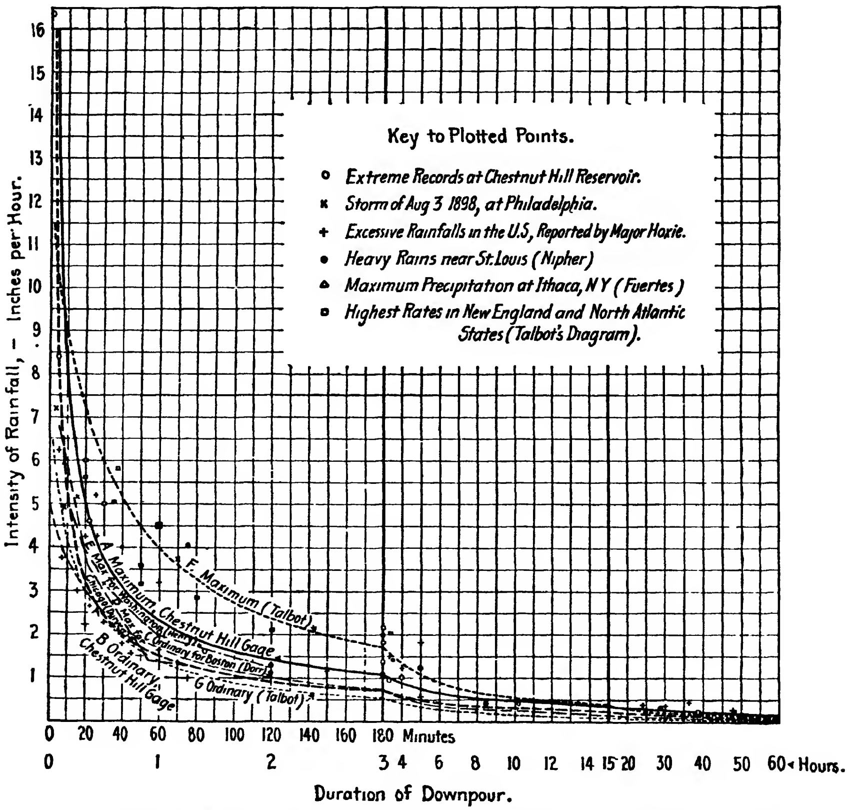


FIG. 5.—Relation between intensity and duration of rainfall; comparison of curves (T. A. S. C. E., Vol 54, 1905)

For ordinary engineering design either of the three "ordinary" curves would be as heavy as there would usually be necessity for considering. In extreme cases, such as important spillways and culverts, it may be necessary to consider rainfalls as heavy as Sherman's "maximum" curve. Such rainfalls may be expected once in 8 or 10 yrs. Such extreme rainfalls as Talbot's "maximum" curve (between 5 min. and 15 hrs., beyond which this curve gives too low results) must be expected about once a century.

Kenneth Allen kept rough memoranda of excessive rainfalls for many years:

of 31 rates noted, 18 of which are for Eastern States, 17 exceed the rates for corresponding intervals by Talbot's "maximum" curve, and, of the latter, but 6 are for periods less than 2 hrs. Rates for shorter periods almost invariably fall within maximum Chestnut Hill curve.

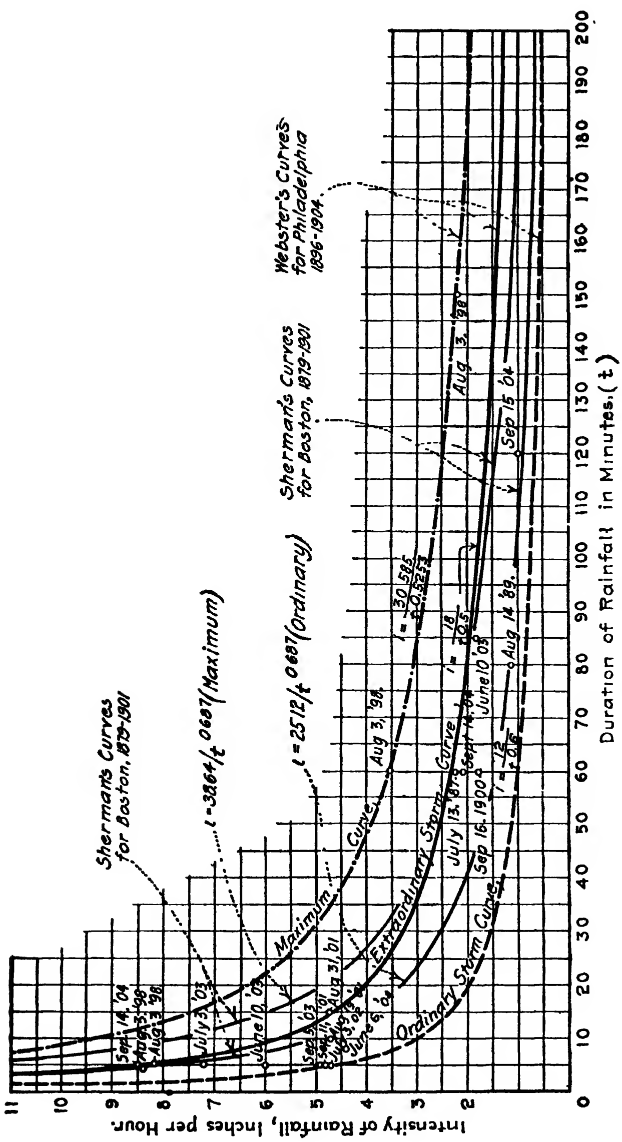


FIG. 6.—Relation between intensity and duration of rainfall. (T. A. S. C. E., Vol. 54, 1905.)

From several recent compilations of heavy rainfalls of uniform intensity, relating to vicinity of New York City, Emil Kuichling adopted $i = 120 \div (t + 20)$.

CHAPTER II

EVAPORATION

Evaporation is used to denote both the process of vaporization and the quantity of water vaporized and diffused into the atmosphere from land and water surfaces, and frequently includes transpiration, deep seepage and other losses. (A. F. Meyer, Trans. Am. Soc. C. E., Vol. 79, 1915.)

Owing to great humidity, evaporation while rain or snow is falling is small. Snow may remain on ground a long time, and, as a rule, is evaporated to greater degree than rain, especially during periods of sunshine and warm winds following storms. Evaporation from different areas also differs greatly; from forest-covered soils it is relatively small; from open plains relatively large.

Evaporation depends on: (*a*) Quantity of rainfall. (*b*) Distribution of rainfall. (*c*) Extent of the water surface. (*d*) Temperature. (*e*) Barometric effect, which is slight outside of effect of altitude on pressure. A liquid may exist at any temperature only when the pressure upon it is greater than the pressure of its vapor at that temperature. Tate says: "Other things being equal, the evaporation is nearly inversely proportional to the atmospheric pressure." (*f*) Mean daily atmospheric pressure. (*g*) Mean annual atmospheric pressure. (*h*) Wind movement. (*i*) Inclination and geological character of the watershed. (*j*) Extent of forest area, including sprout land. (*k*) Extent of swampy and marshy land. (*l*) Extent of cultivated land. (*m*) Humidity. (*n*) Extent of watershed. Temperature is one of the most important.

Laws of Evaporation. There are 14 general laws of evaporation. 1. For rainfall distributed uniformly throughout the year, evaporation increases proportionally with rainfall. Distribution in showers, downpours, long drizzling rains, and depths and promptness of melting of snowfalls, are important. 2. Heavy winter snow and light summer rainfall together produce a small annual evaporation, and conversely. 3. The greater the watershed, the greater will be the evaporation. 4. The greater the area of water surface on the watershed, the greater will be the evaporation. 5. Evaporation varies nearly inversely as the atmospheric pressure, or nearly directly as the altitude of the watershed. (This is questioned by some authorities.) 6. The rate of evaporation is nearly proportional to the difference of temperatures indicated by the wet bulb and the dry bulb thermometers. 7. The capacity of the atmospheric air for moisture is approximately doubled for each 20° F. increase in atmospheric temperature. From data on Croton, Pequannock and Sudbury sheds, T. Merriman deduces the law: For each degree increase in temperature, the rainfall evaporated will be increased by very nearly 2 per cent. 8. Evaporation varies nearly directly as the wind movement. 9. Evaporation from a

watershed varies approximately inversely as the square root of the sine of the angle of its average inclination. "Average inclination" is the difference in altitude between the highest and the lowest point, divided by the diagonal of a square of area equivalent to the watershed. (Questioned by some authorities.) 10. Evaporation from a watershed varies nearly as the extent of the surface exposed. The extent of the surface exposed is nearly proportional to area of watershed divided by the cosine of the angle of its average inclination. (Also questioned by some authorities.) 11. Time is an important factor in evaporation, and the shorter the time that it takes water to run off, the less will be the evaporation. Evaporation is comparatively slight in the water-courses. Evaporation varies inversely as the time, and time in turn varies nearly directly as the square root of the sine of the angle of inclination. 12. Evaporation varies nearly inversely as the porosity of the materials covering the watershed, or rather as the depth of the surface of saturation. 13. Evaporation varies approximately with the extent of cultivated land on the watershed. 14. Evaporation varies inversely with the extent of forest and sprout area on the watershed.

Vermeule's Evaporation Formula. In formulas below, for monthly evaporation, e = monthly evaporation, inches; $f = (0.05\,t - 1.48)$, where t is the mean monthly temperature, deg. F., r = monthly rainfall, in. For

Month	Formula	Month	Formula
Jan.........	$e = f(0\ 27 + 0\ 10r)$	July.........	$e = f(3.00 + 0.30r)$
Feb.........	$e = f(0\ 30 + 0.10r)$	Aug	$e = f(2\ 62 + 0\ 25r)$
Mar.........	$e = f(0\ 48 + 0.10r)$	Sept...... .	$e = f(1\ 63 + 0.20r)$
Apr.........	$e = f(0\ 87 + 0.10r)$	Oct . ..	$e = f(0.88 + 0\ 12r)$
May........	$e = f(1.87 + 0\ 20r)$	Nov .	$e = f(0\ 66 + 0\ 10r)$
June... .. .	$e = f(2.50 + 0\ 25r)$	Dec	$e = f(0.42 + 0\ 10r)$

yearly evaporation, $E = F\,(15.50 + 0.16\,R)$, where E = yearly evaporation, in.; $F = (0.05\,T - 1.48)$; T = mean yearly temperature, deg. F.; R = yearly rainfall, in.

This formula was deduced from New Jersey conditions (1894 report, New Jersey Geological Survey), and has the advantage of allowing for the monthly variations in rainfall. Constants are based on mean temperatures of watershed.

FitzGerald's Evaporation Formula. (T. A. S. C. E., Vol. 15, 1886, p. 589.)

$$E_h = \frac{(S - F_a)(1 + V/2)}{60} \qquad F_a = S - \frac{0.480(T_a - T_e)b}{689 - T_e}$$

b = hight of barometer, in. of mercury; E_h = evaporation per hr., in.; S = maximum force of the vapor in inches of mercury, corresponding to temperature of the water; F_a = force of atmospheric vapor, in inches of mercury; V = velocity of the wind, miles per hr.; T_a = temperature of the air, deg. C., by dry thermometer; T_e = temperature of evaporation in deg. C., by wet thermometer. No difference in evaporation is found between a pan in the sun and one in the shade. The depth of water has no other influence on evaporation than that due to its effect on the temperature of the water. Wind factor = $1 + 0.67V^{\frac{2}{3}}$. Ordinary barometric changes are so slight that they may be

Table 14. Evaporation and Temperatures of Water Surfaces. FitzGerald*

Month	Average temperature of water, deg, Fahr	Evaporation, inches
January	32.8	1.0
February	32.4	1.0
March	36.4	1.7
April	46.5	3 0
May	58.6	4.5
June	67.9	5.5
July	72 3	6.0
August	71 3	5.5
September	65.6	4.1
October	53 6	3.2
November	42.8	2.2
December	34 3	1.5
Total for average year	.. . ..	39 2

disregarded so far as influence on natural evaporation goes. For the correct wind factor, measure the velocity near the water surface, not at some distance from it, nor far above it. FitzGerald's complete formula for evaporation is:

$$E_h = [0.014(S - F_a) + 0.0012(S - F_a)^2](1 + 0.67V^{\frac{1}{2}})$$

The simpler formula is sufficiently accurate for most practical purposes.

Evaporation is influenced by the wind, as shown by the following observations:

Wind velocity, mi. per hr	0	5	10	15	20	25	30
Relative evaporation observed	1.0	2.2	3.8	4 9	5.7	6 1	6 3
Relative evaporation computed by FitzGerald's formula	1.0	2.8	3.7	4.4	5.0	5.6	6.2

Wind Factor. See paragraph above for FitzGerald's. Russel found $(1 + V/4)$, for velocities up to 15 or 20 mi. per hr.; Bigelow, $(1 + V/11)$; A. F. Meyer uses $(1 + V/10)$. Wind velocities, as recorded by the Weather Bureau (see Table 209, p. 627) are those about 30 ft. above the ground and are about 3 times more than those at the surface.

U. S. Weather Bureau Formula.† Observations by the U. S. Weather Bureau at Reno, Nev., Aug. 1 to Sept. 15, 1907, show that the locations of the pans relative to the water of the reservoir are important in measuring total evaporation. Readings on pans distant from a reservoir cannot be transferred to the water surface without the utmost caution.

Study of observations at Abbassia, Boston, Fort Collins and Nakuss showed the constants of Dalton's evaporation formula to be very inconsistent; it was inferred therefore that the formula is not satisfactory. The following formula, based on observations, is proposed:

$$E_h = Cf(h)E\frac{de}{ds}(1 + Aw)$$

$Cf(h)$ = a variable, a function of the hight of pan.

* Based on experiments at Chestnut Hill reservoir, Boston, 1875–1890. The table above and other experiments indicate that each 9° F change in water temperature corresponds to about 1 in. per month of change in amount of evaporation Rate of evaporation was measured at 4 places in Maine by U. S Geol Survey, July 1, 1905 to Nov. 7, 1908 (Water Supply Paper #279). The annual evaporation was substantially 26 in.

† Monthly Weather Review, Feb., 1908.

E = vapor pressure, corresponding to the dew point of air.
de/ds = ratio of increase of vapor pressure to increase of temperature, Centigrade.
w = wind velocity, kilometers per hr.
A = a wind constant.

The size of pan makes an insignificant difference in evaporation, under the same local conditions.

Meyer's Formula. For evaporation, at St. Paul, Minn., A. F. Meyer (Trans. A. S. C. E., Vol. 79, 1915, p. 1074) used following formula:

$$E_m = 15\,S\,(1-d)\left(1+\frac{V}{10}\right)$$

in which E_m = evaporation, in inches per month;
S = maximum vapor pressure, in inches of mercury, at monthly mean air temperature;
d = relative humidity of atmosphere to water vapor;
V = wind velocity, in miles per hour, as measured by the Weather Bureau, approximately 30 ft. above the general level of the surrounding country.

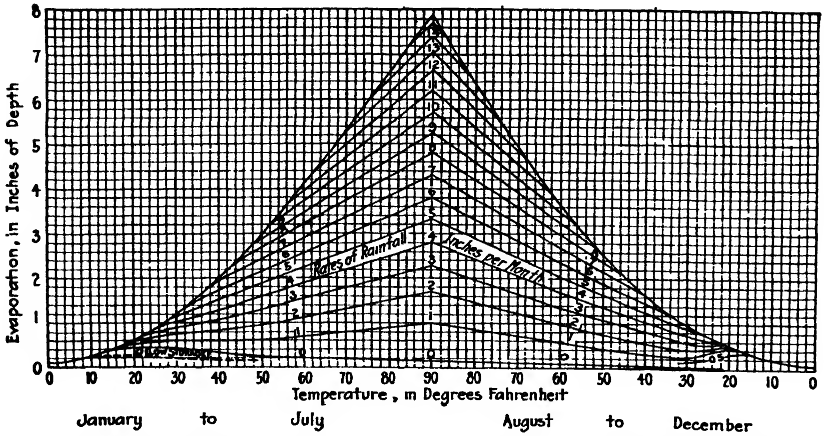


FIG. 7.—Evaporation from land for various temperatures and rainfall rates.
(Meyer, Trans A S C E, Vol 79, 1915, p 1099)

"In winter, when monthly temperature reaches 20° F., practically all precipitation occurs as snow; consequently, evaporation for temperatures below 20° is no longer dependent on precipitation after ground has been covered with snow, but entirely on temperature. Full evaporation, corresponding to given monthly temperature, is usually possible throughout winter. After temperature rises above 20°, in spring, evaporation again depends largely on available moisture, as determined mainly by precipitation. Nevertheless, a considerable constant evaporation is still possible, irrespective of precipitation, because a certain quantity of snow and ice is almost always on ground while monthly temperature ranges from 20° to 35° F. After snow has disappeared, there will still be a relatively large constant evaporation, irrespective of rainfall, unless winter precipitation has been distinctly deficient."

An *evaporation coefficient* must be applied to values of evaporation, in inches depth per month, from Fig. 7, to reduce these quantities to actual evaporation from

the given watershed. This coefficient ranges from about 0.95 to 1.25 for most watersheds of the Northwest, and for similar ones elsewhere. The coefficient to be used depends on topography, vegetal cover, soil, subsoil, humidity, and wind. An extremely high coefficient of evaporation would result from flat topography devoid of vegetation, moderately pervious, shallow soil underlain with impervious subsoil or rock, low humidity, and high wind velocity. An extremely low coefficient would result from rugged topography, bare scanty soil underlain with rock, high humidity, and low wind velocity."

Lowcock's Formula. S. R. Lowcock, in a paper before the English Association of Water Engineers,* concludes that there is no definite relation between evaporation and rainfall, although many investigators prior to 1875 expressed evaporation in terms of rainfall. A small body of water conforms more nearly to air temperature than does a large one; therefore laboratory experiments with small vessels lead to errors. In the Netherlands it was proved that the maximum rainfall and maximum evaporation rarely coincided. Twenty-years' records at Camden Square indicated a relation between evaporation and sunshine. It was also shown that rainfall and evaporation bore no inverse relationship. Barometric pressure and wind velocity have a negligible effect. From 4-year records, the following formula gives results within 8 per cent., being high for summer and low for winter: Evaporation (in. per month) = mean maximum sunshine temperature in deg. F. × duration of sunshine in hours × 0.0001254. The factor is probably only true for England and places of similar latitude and temperature. Tank records are on the safe side. FitzGerald's tests with a tank at Chestnut Hill reservoir, Boston, support the contention that radiant heat has a far greater effect on evaporation than heat transmitted by contact.

Dalton's Formula.† Evaporation from the free surface of water is given by the following equation by Dalton:

$$E_h = 1.80 \times c \times S\left(\frac{1-d}{b}\right)$$

where c = empirical constant which depends on the air circulation over the water.

S = maximum tension of water vapor at the temperature of the evaporating water, in. of mercury.

d = relative humidity of atmosphere to water vapor.

b = reading of barometer, in. of mercury.

For quiet air, $c = 0.55$; for moderately agitated air, $c = 0.71$; for heavy wind, $c = 0.86$.

Maxima tensions, S, of water vapor for different temperatures are given in Table 17, page 37. For moderately agitated air at temperature of 50° F., and with barometer at 29.53 in., and with 60 per cent. relative humidity in the air, $E_h = 1.80 \times 0.71 \times 0.36\left(\frac{1-0.6}{29.53}\right) = 0.0062$ in. per hr.

Lueger assumes for the dry period an average evaporation of 0.16 to 0.39 in. per day, according to climate. Other observers have found that in the temperate zone, with a yearly average temperature of 50° F., the yearly

* Surveyor, Dec. 24, 1909.

† König: Wasserleitungen und Wasserwerke, 1907, p. 228.

evaporation from a free surface is 3 ft. At a temperature of 77° F., the daily evaporation might be 0.4 in. which gives for each sq. mi. of free water surface a daily loss of 929,000 cu. ft.

Temperature and Wind Relation, Dalton's Formula.* In the tropics, the average annual temperature is 25° C. = 77° F.; from 23½ to 40° of north or south latitude, average temperature is 20° C. = 68° F.; from 40 to 50° latitude, 12.5° C. = 54.5° F.; from 50 to 60° latitude, 5° C. = 41° F.; from 60 to 66½° of latitude (polar circles), 2.5° C. = 36.5° F. and from the Polar Circles to the Poles, − 2° C. = 28.4° F. If there is normally a strong wind blowing (c = 0.86) so that 75 per cent. represents the relative humidity of air adjacent to water,

$$E_h = 1.80 \times 0.86 \times S(1 - 0.75) \div 29.92$$

Table 15. Relation of Average Yearly Temperature to Yearly Evaporation*

Computed from Dalton's formula (Barometer at 29.92 in.)

Temperature	Centigrade......	25	20	12	5	2.5	− 2
	Fahrenheit.......	77	68	54	41	36	28
Yearly evaporation	mm.	2660	1965	1180	738	630	452
	in	105	77	46	29	25	18

Within the range of annual variation of temperature prevailing throughout the northwest U. S., the rate of evaporation will vary about 700 to 1200 per cent., due to temperature changes alone, according to A. F. Meyer (Trans. A. S. C. E., Vol. 79, 1915).

Humidity. The amount of evaporation from the free water surfaces of arid lands is considerably greater, especially in warm countries, than evaporation from the sea, situated in the same latitude, since the air possesses much lower relative humidity over the heated lands than over the sea. In the temperate zones, the values of relative humidity inland and at sea approach each other.

New York State Barge Canal, in Kuichling's report, is estimated to lose daily 0.30 in. by evaporation. He adds 10 per cent. to this, as a provision for consumption by aquatic plants, giving a total of 0.33 in.

From Reservoirs.† Evaporation is subject to great variation due to the difference in mean depth, temperature, winds and relative humidity. Evaporation losses must be considered in all computations of reservoir capacity. They may reduce the reservoir capacity 50 per cent. per annum. Careful measurements in a floating pan at the Sweetwater Dam in California showed an annual loss of 54 in.; the loss was 2 in. in Jan., and 8 in. per month for July and August. This amounts to an annual loss of 15 per cent. of the stored water, and as the reservoir must hold 2 yrs.' supply to tide over dry years, the evaporation will amount to 30 per cent. of the water impounded. This leaves 70 per cent. as the available capacity of the reservoir. At Cuyamaca reservoir on an adjacent watershed, the average annual evaporation loss for 9 yrs. was 56.7 in. This amounts to a 26 per cent. yearly loss, considerably larger than for Sweetwater, probably due to the much greater exposed surface per volume stored. These instances show the advantages of deep reservoirs and high dams for the effective conservation of water.

* König: "Wasserleitungen und Wasserwerke," 1907, p 17.

† Schuyler: Reservoirs for Irrigation, Water-Power and Domestic Water Supply, 1908, p 237.

Variation in United States. Evaporation in the U. S. varies from 18 to over 100 in., annually, being greatest in the arid regions, and least at high

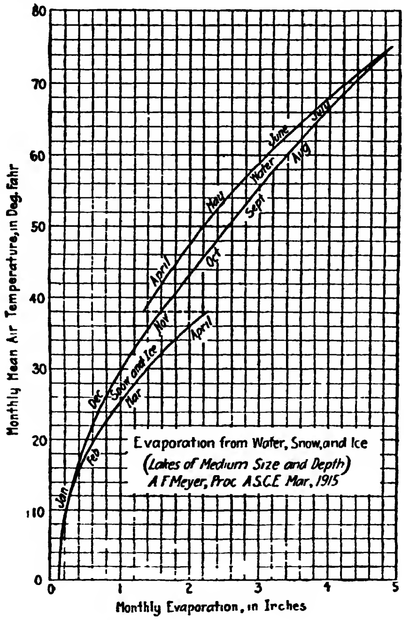


FIG. 8

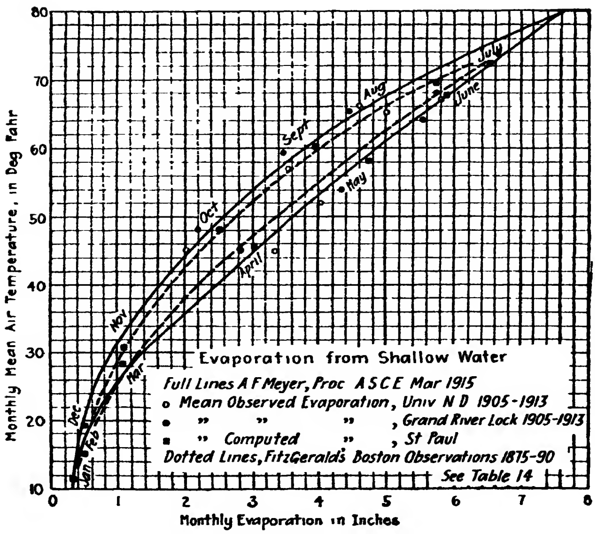


FIG. 9.

altitudes in the north. From tests, U. S. W. B. concludes that if evaporation from a large water surface is 1, that from a pan of 2 ft. diam. is at rate of 1.75; diam. 4.5 ft., rate 1.50; diam. 6 ft., rate 1.30.

Table 16. Evaporation from Water Surfaces in Inches

Locality (also position of test pan if known)	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sept	Oct.	Nov.	Dec.	12 Mos.	Year	Remarks, size of test pan and altitude at which placed
Salton sea, Cal., 1500 ft. inland	5.1	7.4	12.5	15.8	19.0	21.5	22.2	18.5	15.5	13.2	7.5	6.4	164.5	1909–10	D = 2, ground
Salton sea, Cal., 1500 ft. inland	7.1	9.4	13.2	17.2	20.5	23.5	25.0	21.2	19.5	17.2	10.0	9.6	193.4	1909–10	D = 2, 40 ft. high
Salton sea, Cal., 500 ft. at sea	3.6	5.0	6.8	9.0	11.0	13.5	14.8	12.5	12.4	9.2	6.2	4.7	108.6	1909–10	D = 4, 2 ft. high
Salton sea, Cal., 500 ft. at sea	5.1	7.2	9.0	10.5	13.0	16.8	18.6	15.0	15.2	12.2	8.1	7.0	137.7	1909–10	D = 4, 45 ft. high
Salton sea, Cal., 7500 ft. at sea	3.4	5.1	7.0	8.8	10.5	13.0	14.0	12.2	12.1	9.2	6.0	5.2	106.4	1909–10	D = 4, 2 ft. high
Salton sea, Cal., 7500 ft. at sea	4.7	7.4	9.5	11.5	14.0	16.8	18.0	15.3	15.4	13.0	7.5	7.0	140.0	1909–10	D = 4, 45 ft. high
Indio, Cal., 15 mi. north of Salton sea	3.2	5.1	7.5	12.0	15.8	16.1	16.3	13.8	12.4	8.9	5.2	3.0	119.3	1909–10	D = 6, ground
Indio, Cal., 15 mi. north of Salton sea	5.5	8.8	12.1	19.2	25.1	26.7	27.2	23.0	21.1	16.8	9.4	5.2	200.4	1909–10	D = 6, 10 ft. high
Mecca, Cal., 0.5 mi. north of Salton sea	2.9	5.0	8.1	10.9	12.7	14.2	15.2	13.2	10.3	8.2	4.1	3.0	107.8	1909–10	D = 6, ground
Mecca, Cal., 0.5 mi. north of Salton sea	5.5	8.8	11.9	17.0	21.3	21.6	22.6	20.4	16.9	12.4	7.2	4.6	170.0	1909–10	D = 6, 10 ft. high
Brawley, Cal., 20 mi. south of Salton sea	3.0	5.0	8.0	10.7	13.8	13.7	14.1	11.3	10.2	7.0	4.1	2.7	103.6	1909–10	D = 6, ground
Brawley, Cal., 20 mi. south of Salton sea	5.3	8.0	11.0	16.0	21.6	20.2	21.0	21.2	16.3	11.6	7.0	4.6	163.7	1909–10	D = 2, 10 ft. high
Mammoth, Cal., 40 mi. southeast of Salton sea, in desert	4.2	5.7	9.0	12.0	15.5	16.8	18.0	13.7	12.2	9.5	5.3	3.7	125.5	1909–10	D = 6, ground
Mammoth, Cal., 40 mi. southeast of Salton sea, in desert	6.5	8.9	11.6	17.1	22.0	24.2	25.7	18.2	17.0	14.7	8.1	5.1	179.1	1909–10	D = 2, 10 ft. high
Calexico, Cal.	...	..	..				14.5	12.7	10.3	7.5	4.8	4.5	..	1903	
Calexico, Cal.	4.4	6.3	8.9	9.6	10.9	13.9	12.5	11.0	8.6	8.8	5.4	3.5	103.6	1904	
Calexico, Cal.	2.7	1.7	4.4	4.7	8.4	12.9	10.4	8.5	7.8	6.8	3.2	3.4	75.1	1905	All inland tests. Tanks, 22 to 36 in.; diam., 30 in. deep, set 29 in. in ground. S. Fortier, U. S. Dept. of Agriculture, Bull. 177, 1907.
Pomona, Cal.	.	...		.			9.1	9.4	6.3	6.6	4.2	2.5	.....	1903	
Pomona, Cal.	2.8	2.6	3.7	5.0	6.5	8.2	9.3	9.4	7.2	5.4	4.0	2.9	67.0	1904	
Pomona, Cal.	1.9	1.6	3.7	4.1	6.0	7.7	8.9	9.0	7.4	5.3	4.0	2.9	62.8	1905	
Tulare, Cal.	.	.	.					12.3	9.1	6.3	3.5	2.0	.....	.	
Tulare, Cal.	1.5	3.0	3.8	4.6	8.0	10.7	12.2	12.0	8.7	4.3	3.9	2.0	74.5	1904	
Tulare, Cal.	1.5	2.5	2.9	3.3	6.8	10.3	11.1	7.6	6.3	4.8	3.9	2.0	62.9	1905	
Chico, Cal.	0.5	1.1	1.1	3.0	5.6	6.8	8.3	7.7	4.8	2.2	1.6	1.0	43.5	1904	
Chico, Cal.	0.1	1.2	3.8	4.8	6.8	9.1	10.0	9.6	7.5	5.7	3.4	1.5	63.5	1905	
Berkeley, Cal.	..	..	.	....	..	..	.	.		2.8	1.4	1.0	......	1904	
Berkeley, Cal.	1.0	1.4	2.1	3.1	4.7	5.7	5.5	5.1	4.6	4.3	2.7	1.3	41.6	1905	
Lake Tahoe, Cal. (elev. 6200 ft.)	1.8	1.8	1.8	2.0	3.0	4.2	6.2	7.1	6.2	3.6	2.6	2.0	42.2	1909–10	D = 4, 2 ft. high
Lake Tahoe, Cal. (elev. 6200 ft.)	1.8	1.8	1.8	2.2	3.5	5.0	6.5	7.4	7.8	4.3	3.1	2.2	47.4	1909–10	D = 3, 10 ft. high
Lake Tahoe, Cal.	3.8	3.1	2.2	1.3	1.1	0.9	1.2	1.8	2.4	3.3	4.4	4.9	30.5	1900–06	With some deductions E. N., Feb. 29, 1912.
Salt River, Granite Reef, near Phoenix, Ariz.	4.6	4.8	6.2	9.0	11.5	13.5	14.2	14.2	13.8	11.3	7.4	4.6	115.2	1909–10	D = 4, ground
Salt River, Granite Reef, near Phoenix, Ariz.	5.3	5.5	7.2	10.5	13.5	15.5	16.6	16.4	16.7	14.2	9.8	5.6	136.8	1909–10	D = 3, 10 ft. high
Salt River, Granite Reef, near Phoenix, Ariz.	4.2	4.4	5.2	7.0	9.5	12.0	12.8	12.5	11.0	8.3	6.6	4.2	97.7	1909–10	D = 4, on raft
Holbrook, Ariz.	.....	....	. .	...	....	. ..	..	6.6	4.6	4.3	1.4	0.2	......	1905	Lat. 34.5° N., long. 110° W., elev., 5072 ft., annual rainfall, 8.99 in.
Holbrook, Ariz.	....	1.2	3.3	5.1	6.9	8.6	7.4	6.2	5.1	3.4	.....	.....		1906	
Holbrook, Ariz.	...	1.2	2.3	3.8	4.0	3.9	6.4	6.4	4.6	4.0	2.5	2.4	48.6	1907	
Holbrook, Ariz.	3.4	3.4	2.8	2.8	4.9	5.2	6.8	5.2	4.1	3.5	3.2	3.3	45.4	1908	
Holbrook, Ariz.	4.1	3.3	2.0	2.8	4.3	6.8	7.8	4.1	3.6	3.1	2.1	1.4	......	1909	

Eng. News, June 16, 1910, unless otherwise credited. D = diam. of test pan, ft.

Continued on next page

Table 16. Evaporation from Water Surfaces in Inches—*Cont'd.*

Locality (also position of test pan if known)	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	12 Mos.	Year	Remarks, D = diam. of test pan, ft.
Rio Grande, Elephant Butte, N. Mex.	**2.5**	**2.8**	**4.5**	**8.0**	**11.5**	13.4	11.6	10.5	8.6	6.8	3.9	**3.0**	87.0	1909–10	D = 4 ft., ground
Rio Grande, Elephant Butte, N. Mex.	**2.8**	**3.0**	**5.2**	**10.2**	14.8	16.5	13.7	12.3	10.3	8.3	4.9	**3.2**	105.3	1909–10	D = 3, 10 ft. high
Carlsbad, N. Mex.	**5.0**	**5.5**	8.9	11.7	12.9	12.4	12.0	11.0	9.8	7.6	5.5	**5.0**	107.2	1909–10	D = 4, ground
Carlsbad, N. Mex.	**6.0**	**7.0**	9.9	12.4	13.8	13.5	13.0	10.8	9.2	7.0	6.4	**6.0**	115.0	1909–10	D = 3, 10 ft. high
Carlsbad, Alfalfa field	**5.0**	**5.2**	**9.0**	11.1	11.0	9.1	10.6	9.3	7.8	5.9	5.4	**5.0**	94.4	1909–10	D = 4, ground
Carlsbad, Alfalfa field	**6.5**	**7.0**	**10.5**	13.0	16.4	14.7	14.5	11.5	10.2	8.5	7.2	**6.5**	126.5	1909–10	D = 3, 10 ft. high
Pecos river, Lake Avalon	**4.5**	**4.5**	5.5	7.4	10.1	11.0	12.9	**12.0**	**9.5**	**7.0**	**5.8**	**4.5**	94.5	1909–10	D = 4 ft., floating
Pecos river, Lake Avalon	**6.5**	**7.0**	8.7	13.8	13.7	18.4	17.8	15.8	12.3	**9.5**	**7.5**	**6.8**	137.8	1909–10	D = 3, 10 ft. high
Truckee-Carson, Fallon, Nev.	**1.8**	**1.8**	**2.2**	**3.2**	**5.2**	7.9	9.9	8.7	5.1	3.4	**2.5**	**2.0**	53.6	1909–10	D = 4, floating
Truckee-Carson, Fallon, Nev.	**2.0**	**2.2**	**3.0**	**4.5**	**7.5**	13.0	15.8	14.2	8.7	5.3	**3.2**	**2.2**	81.9	1909–10	D = 3, 10 ft. high
Klamath, Ore., Ady	**0.5**	**1.2**	3.6	6.6	7.2	7.0	8.0	9.2	6.1	2.5	1.0	**0.5**	53.4	1909–10	D = 4, floating
Klamath, Ore., Ady	**1.0**	**1.8**	4.8	8.2	9.1	10.0	11.1	12.4	7.0	4.7	2.8	**1.2**	74.1	1909–10	D = 3, 10 ft. high
Klamath, Ore., Ady	**1.0**	**2.0**	5.0	8.8	9.8	11.3	11.9	13.3	7.9	5.0	2.8	**1.5**	80.4	1909–10	D = 3, 20 ft. high
Hermiston, Ore., Umatilla project	**1.2**	**1.2**	**3.0**	7.3	7.9	9.5	12.0	11.1	7.4	3.9	**2.0**	**1.5**	68.0	1909–10	D = 4, raft
Hermiston, Ore., Umatilla project	1.5	**1.8**	**4.2**	**9.3**	11.4	13.8	17.5	16.9	10.1	6.1	**3.0**	**1.8**	97.3	1909–10	D = 3, ground
Hermiston, Ore., Umatilla project	**2.0**	**2.2**	**5.5**	9.4	10.9	13.5	17.3	16.0	10.6	7.3	**4.0**	**2.5**	101.1	1909–10	D = 3, 25 to 65 ft.
Yakima river, Lake Kechess, Wash.	**0.5**	**0.5**	**1.2**	2.6	3.8	5.5	5.9	5.5	4.4	1.5	**0.8**	**0.5**	32.8	1909–10	D = 3, 10 ft. high
N. Yakima, Wash., U. S. Reclamation project	**1.8**	**2.5**	**6.2**	7.9	8.4	8.9	10.7	9.4	5.5	3.2	**2.0**	**1.5**	68.0	1909–10	D = 4, ground
N. Yakima, Wash., U. S. Reclamation project	**2.0**	**4.2**	**8.2**	9.9	10.2	11.1	12.5	11.6	7.8	4.0	**2.5**	**2.0**	86.1	1909–10	D = 3, 10 ft. high
Snake river, Idaho, Minidoka Dam	**2.2**	**2.5**	**4.0**	**7.0**	11.2	12.3	15.0	**13.5**	**11.0**	**8.5**	**5.8**	**3.5**	96.5	1909–10	D = 3, 10 ft. high
Payette-Boise, Idaho, Deer Flat	**1.5**	**2.2**	**4.0**	**7.2**	10.7	11.0	11.2	11.8	**9.8**	5.4	2.7	**1.5**	79.0	1909–10	D = 3, ground
Payette-Boise, Idaho, Deer Flat	**1.8**	**2.5**	**5.5**	**9.5**	11.8	12.0	11.7	11.5	**10.5**	5.0	4.2	**1.8**	87.8	1909–10	D = 3, 20 ft. high
Payette-Boise, Idaho, Deer Flat	**2.0**	**2.8**	**4.2**	**6.0**	7.9	9.6	10.6	12.2	**9.2**	5.4	5.5	**2.0**	77.4	1909–10	D = 4, floating
Payette-Boise, Idaho, Deer Flat	**2.0**	**3.0**	**4.5**	7.0	9.9	10.8	12.5	12.5	**10.5**	5.8	3.6	**2.0**	84.1	1909–10	D = 4, 20 ft. high
Nebraska Interstate Canal, Dutch Flats.	**1.8**	**1.8**	**3.0**	**4.5**	**6.2**	8.0	11.0	9.4	7.4	5.6	**4.0**	**3.0**	65.7	1909–10	D = 4, ground
Nebraska Interstate Canal, Dutch Flats.	**2.0**	**2.2**	**3.5**	**6.0**	**8.5**	11.0	14.7	12.7	10.0	7.6	**5.2**	**3.0**	86.4	1909–10	D = 3, 10 ft. high
California, O., Cincinnati Filter	**1.0**	**1.5**	**2.5**	4.1	5.1	6.2	7.2	7.3	5.6	**3.0**	**1.5**	**1.0**	46.0	1909–10	D = 4, floating
California, O., Cincinnati Filter	**1.2**	**2.0**	**3.5**	6.6	7.7	8.3	9.7	8.9	6.7	**4.0**	**2.0**	**1.2**	61.8	1909–10	D = 3, 10 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.5**	**2.2**	5.4	6.4	7.5	7.0	7.3	5.6	**4.0**	**2.2**	**1.5**	52.1	1909–10	D = 4, floating
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	6.6	7.6	8.7	9.6	10.4	7.1	**5.0**	**2.8**	**1.8**	65.2	1909–10	D = 2, 10 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	6.9	7.7	8.9	9.8	10.9	8.9	**5.0**	**2.8**	**1.8**	66.2	1909–10	D = 2, 20 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	7.1	7.8	9.0	10.2	11.0	8.9	**5.0**	**2.8**	**1.8**	69.4	1909–10	D = 2, 40 ft. high
Boston, Mass., Chestnut Hill	1.0	1.0	1.7	3.0	4.5	5.5	6.0	5.5	4.1	3.2	2.2	1.5	39.2	1875–90	T.A.S.C.E., Vol. 27, 1892.
Mt. Hope Res., Rochester	0.5	0.5	1.3	2.6	3.9	4.9	5.5	5.3	4.2	3.2	1.4	1.1	34.5	1891–98	Turneaure & Russell, Public Water Supplies, 1908, p. 56.
Lee Bridge, England	0.8	0.6	1.1	2.1	2.8	3.1	3.4	2.8	1.6	1.1	6.7	0.6	20.6	1860–73	

* Figures in heavy face type were obtained by interpolation (Abstract of Data #4, U. S. Dept. of Agr. Weath. Bur.). Observed evaporations, no correction made for wind, temperature, vapor pressure, or size of pan.

Table 17. Maximum Tension of Water Vapor for Different Temperatures

Temperature °C	Temperature °F	Vapor tension mm., mercury	Vapor tension in., mercury	Temperature °C	Temperature °F	Vapor tension mm., mercury	Vapor tension in., mercury
−20	−4 0	0.927	0.04	16	60.8	13.536	0.53
−18	−0.4	1.100	0.04	18	64.4	15.357	0.60
−15	+5 0	1.400	0.06	20	68.0	17.391	0.68
−12	10.4	1.780	0.07	22	71 6	19.659	0.77
−10	14 0	2.093	0 08	24	75.2	22.184	0.87
− 8	17.6	2 455	0 10	25	77.0	23.550	0.93
− 5	23.0	3.113	0.12	26	78.8	24.988	0.98
− 3	26.6	3.644	0 14	28	82 4	28.101	1.11
0	32.0	4 600	0 18	30	86.0	31.548	1.24
+ 2	35 6	5 302	0 21	32	89.6	35.359	1.39
4	39.2	6 097	0.24	33	91.4	37.411	1 47
5	41.0	6 534	0 26	34	93.2	39.565	1.56
6	42.8	6.988	0.28	35	95 0	41.827	1.65
8	46.4	8.017	0.32	36	96 8	44.201	1.74
10	50.0	9 165	0.36	37	98.6	46.691	1 84
12	53.6	10.457	0.41	38	100 4	49.302	1 94
14	57.2	11 908	0 47	39	102 2	52.039	2 05
15	59.0	12 699	0.50	40	104.0	54.906	2.16

(From König, Wasserleitungen und Wasserwerke, 1907, p 229)

Rate of evaporation from soils depends upon: (1) Amount of precipitation; (2) moisture in the surface soil; (3) temperature of the air; (4) force of the wind; (5) proximity of the surface of saturation to the surface of the ground; (6) the slope and smoothness of the ground. The most complete observations are those made by Gilbert and Laws, in England, from 1870 to 1890, from laboratory experiments on somewhat heavy soils, uncropped, made in tanks wherein the sums of the percolation and evaporation are equal to the rainfall.

Table 18. Relation of Soil Evaporation and Percolation to Rainfall

Month	Average monthly rainfall 1870–90, in.	Evaporation: Average, in.	Evaporation: Per cent of yearly total	Evaporation: Per cent. of monthly rainfall	Percolation at 60-in. depth: Average, in.	Percolation at 60-in. depth: Per cent. of yearly total	Percolation at 60-in. depth: Per cent. of monthly rainfall
Jan.	2 51	0.45	2.7	17.9	2.06	15.2	82.1
Feb.	2.04	0.60	3.6	29.4	1.44	10.4	70.6
Mar...	1.74	0.88	5.3	50.6	0.86	6.3	49 4
Apr	2.21	1.53	9.2	69.2	0.68	5.0	30.8
May	2.28	1.69	10.1	74.2	0.59	4.3	25 8
June ..	2.52	1.92	11.5	76.2	0.60	4.4	23 8
July.. ...	3.03	2.26	13.6	74.6	0.77	5.7	25.4
Aug	2.45	1.95	11.6	79.6	0.50	3.7	20.4
Sept	2.86	2.11	12.6	73.8	0.75	5.5	26.2
Oct	3.20	1.70	10.2	53.1	1.50	11.1	46.9
Nov. .	3.03	0.98	5.9	32.3	2.05	15.1	67.7
Dec	2.42	0.61	3.7	25.2	1.81	13.3	74 8
Year	30.29	16.68	100.0	55.1	13.61	100.0	44.9

Proc. Inst. C. E., Vol. 105, 1891, p. 36.

TRANSPIRATION

Transpiration denotes the water which escapes as vapor from the stomata of leaves, and the process by which such loss of moisture takes place. "Hygroscopic water," that is, the water retained in the vegetable substance produced, is inconsequential.

Transpiration Curve. Base values for total transpiration, in inches of depth, during the growing season on any given watershed, are selected with reference to character of vegetation and length of growing season, giving consideration also to available sunshine. A normal seasonal transpiration of about 9 in. has been assumed for small grains, grasses, and other agricultural crops, 8 in. for deciduous trees, 4 in. for evergreen trees, and 6 in. for small trees and brush. Normal monthly distribution of this total seasonal transpiration is based mainly on temperature. To obtain actual transpiration in any

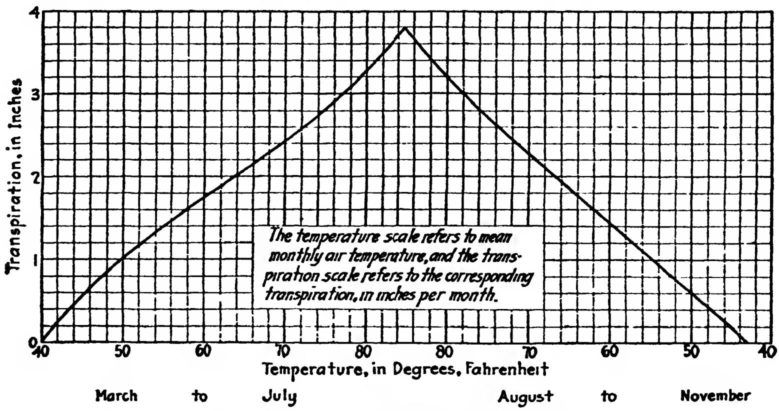


FIG. 10.—Base curve of transpiration.
(Meyer, Trans. A. S C E., Vol 79, 1915, p 1094)

given month, values from the transpiration curve, after being multiplied by a coefficient, must be further modified on the basis of available moisture. Where precipitation minus evaporation for a given month is insufficient to meet normal plant requirements, the ground water is drawn on to a varying extent, depending on character of root system, depth and character of soil, and quantity of surface soil storage, as determined by precipitation minus losses for previous months.*

Table 19. Examples of Results from Transpiration Curve (Fig. 10)

Name of watershed	Year	Total seasonal transpiration, off curve	Name of watershed	Year	Total seasonal transpiration, off curve
Little Fork	1909	8.1	Ottertail	1909	9 3
Little Fork	1910	8 5	St Croix	1907	7 5
Minnesota	1909	10 3	St Croix	1912	9 8
Minnesota	1910	12 3	Tombigbee	1906	20 1
Ottertail	1908	9 8			

In a general way, using mean monthly temperatures for watersheds in Minnesota, curve will give a normal seasonal transpiration of about 10 in. The curve takes into consideration only one factor, temperature. Character and density of vegetation, hours of sunshine, available moisture, etc., all enter in determining transpiration.

* A. F. Meyer, Trans. A. S. C. E., Vol. 79, 1915.

Table 20. Evaporation (Transpiration) from Various Kinds of Vegetation

(HARRINGTON)

Vegetation	Proportion of evaporation from free water surface	Proportion of precipitation*
Sod	1.92	0.96
Cereals	1.73	0.86
Forest	1.51	0 75
Mixed	1.44	0.72
Bare soil	0 60	0 30

*Warm season, May to Sept.

CHAPTER III

RUN-OFF AND STREAM-FLOW

Run-off and Yield. The run-off from a watershed is water that flows in streams which drain the watershed. Yield is the collectible portion of the rain falling on a watershed, gathered in surface or underground channels. A watershed covered with loose gravel and sand will generally show a greater yield than one with a clay cover, as the rainfall sinks into the porous material and is largely protected against evaporation until it drains into the streams.

Factors that modify or control run-off* are numerous and vary widely in different regions. A number are given below. Of some the effects are indicated; of others they will be obvious after a little thought. Some factors mentioned do not apply to small watersheds. On very large watersheds, there may be some counterbalancing of effects.

1. *Precipitation.* (*a*) Rain or snow. (*b*) Amount of each, and the total annual precipitation. (*c*) Distribution throughout the year. (*d*) Intensity or manner of occurrence. (*e*) The character, direction, extent and duration of storms.

2. *Temperature.* (*a*) Variations on the area of watershed. (*b*) Relation of extreme temperatures to the occurrence of precipitation. (*c*) Accumulation of snow and ice caused by low temperature. (*d*) Occurrence of low temperature causing the freezing of the ground at times of heavy rains, resulting in excessive run-off.

3. *Topography.* (*a*) Level, or degree of inclination. (*b*) Character of the area, whether smooth or rough.

4. *Geology.* (*a*) Pervious or impervious. (*b*) If pervious, whether such pervious deposits are (1) shallow or deep; (2) level or inclined; and whether the outlet or point of discharge of the pervious deposits are (3) in the lower valley of the same river, or (4) in valleys of other rivers or in the sea. (*c*) Condition of the channel of stream, whether (1) pervious or impervious; (2) whether the bed contains more or less extensive deposits of sand and gravel, permitting development of more or less extensive underflow.

5. *Condition of the Surface.* (*a*) Extent of vegetation. (*b*) Extent of cultivated areas. (*c*) Nature of vegetation—whether grass-land, crops or forests.

6. *Natural Storage.* (*a*) Nature and extent of surface storage—lakes, ponds, marshes and swamps. (*b*) Nature and extent of ground storage, in gravel, sand and other pervious deposits.

7. *Geography.* (*a*) Size. (*b*) Shape—long and narrow, or short and broad. (*c*) Location relative to prevailing winds. (*d*) Direction relative to path of storms. (*e*) Relation to mountains. (*f*) Distance from the ocean or other

* Bull. 425, 1911, Univ. of Wisconsin, D. W. Mead. (Slightly modified.)

large body of water. (*g*) Snow-capped mountains or glaciers, or high or wooded areas retaining snow late into the warm season.

8. *Character of the Stream and its Tributaries.* (*a*) Slope or gradient. (*b*) Falls and rapids. (*c*) Cross-section of the stream—deep or shallow. (*d*) Arrangement of tributaries—joining the main stream at various points along its course or concentrated in a fan-like arrangement at a common point of discharge.

9. *Artificial Control of the Stream.* (*a*) Dams and storage reservoirs. (*b*) Restrictions by dikes and levees. (*c*) Obstruction by piers, abutments, and other encroachments in the waterway.

10. *Artificial Use of the Stream.* (*a*) Irrigation. (*b*) Water supply. (*c*) Supply of navigation canals. (*d*) Artificial storage and regulation.

11. *Character and Extent of the Winds.* (*a*) Intensity and direction. (*b*) Modification by mountains and forests.

12. *Ice Formation.* (*a*) Modifying the winter flows of the stream. (*b*) Gorges and accompanying floods.

13. *Evaporation.* (*a*) Governed largely by conditions mentioned above. (*b*) Proportion of water surface to total area.

Note. Water diverted by canals, aqueducts, wells, sewerage systems, etc., must be taken into account.

COMPUTING YIELD OR RUN-OFF

Safe possible yield for water supply cannot be computed until the following data are obtained: (1) Catchment area; (2) rainfall: (*a*) minimum year; (*b*) series of dry years; (*c*) average of a long series of years; (3) ground-water diagram of the stream; (4) available storage capacity on the stream; (5) loss by evaporation and percolation; (6) measurements of actual run-off of the stream.

Swamps, Allowance for. Committee of N. E. W. W. Asso., 1914, considered undrained swamps equivalent to 40 per cent. of their areas as water surface and 60 per cent. upland; drained swamps, 30 and 70 per cent., respectively.

Estimates of the available run-off should be based on the minimum for a long series of years, as the water value of a given drainage area during dry years is obviously only the annual run-off of these years. The annual run-off or yield on a given stream occasionally varies nearly 100 per cent. for the same annual rainfall. In New York and New England, the average run-off is about 45 per cent. of the rainfall; 34 yrs.' observations on the Croton watershed show that the average run-off is about 48 per cent., but the years of lowest rainfall show as low as 31 per cent. The dry condition of the ground in times of drought keeps much water in the interstices. From 1868 to 1912, the Croton year of lowest rainfall showed a yield of 603,000 gals. per day per sq. mi.; next driest year, 760,000; average for 45 yrs., 1,084,000. The safe average permissible draft between these limits should be chosen, as

say 850,000. The year of maximum yield, 1901, averaged 1,693,000. Calendar years are referred to in this paragraph.

Formulas for run-off:

Fanning: $Q = 200\ (D_m)^{\frac{5}{6}}$.
Ryves: $Q = C(D_m)^{\frac{2}{3}}$ (India).
Dickens: $Q = C(D_m)^{\frac{3}{4}}$
where D_m = catchment area in sq. mi.
Q = discharge in sec. ft.
C = coefficient.

In regions of maximum recorded rainfall of 3 in. to 6 in. in 24 hrs. coefficient C is·

	Dickens	Ryves
Flat country	200	400 to 500
Mixed country	250	
Hilly country	300	
Maximum rainfall	300 to 350	650

Stream-flow in Average Year. Fig. 11 illustrates a method which may be applied somewhat generally. This mean line was drawn in connection with a study of Housatonic and Ten Mile rivers in western Connecticut. To prepare a similar diagram, compile from records of streams resembling the one to be studied a table of run-offs in c.f.s. per sq. mi. of watershed by months for several calendar years, rejecting those for abnormal precipitation. The longer the records the better. For each calendar year pick out the month of least run-off, and get the average for all the years. Similarly, pick out second driest month, third driest, and so on, and get averages. From these averages plot a curve for each stream of record and then draw the mean line.

Length of Record Necessary for Determining Stream-flow. By study of long records for various streams in U. S., Hydrographer J. C. Hoyt, of Geological Survey, concludes: (1) Values for 1 yr. vary so much from the grand mean that estimates based thereon are liable to be in great error; (2) while 5-yr. period in majority of cases gives values within 10 per cent. of the grand mean, there are periods when the difference may be even 20 or 30 per cent., probability of 5-yr. period comprising both a year of average low and a year of average high water is small; (3) 10-yr. period with few exceptions gives values within less than 10 per cent. of the grand mean; furthermore, there occur in practically each 10-yr. group a year of average low and one of average high water; while this low and high may not be extreme, they give conditions to be expected except in abnormal years, which as a rule occur only once in many years. These deductions do not hold for streams in arid and semi-arid regions, where range is very great from year to year. E. N., Apr. 23, 1908.

The average run-off of Croton river for 40 yrs., 1868 to 1907, was 400 mgd.; from 1869 to 1886, 18 yrs., flow averaged 346 mgd.; and for the next 18 yrs., 449 mgd. This shows the futility of attempting to use even reasonably long run-off records (5 to 15 yrs.) as a basis for computing the safe yield of a watershed, unless these records are compared with longer ones of similar watersheds,

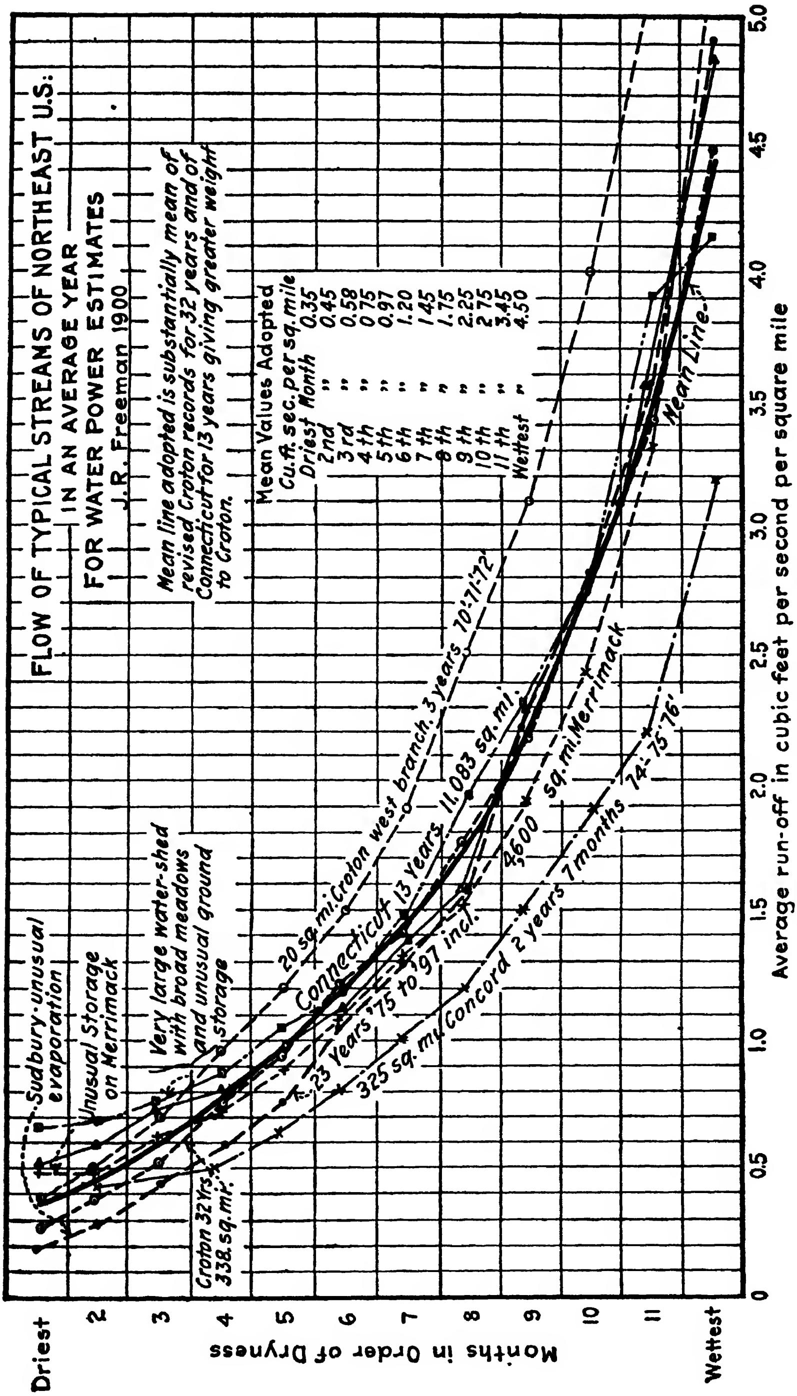
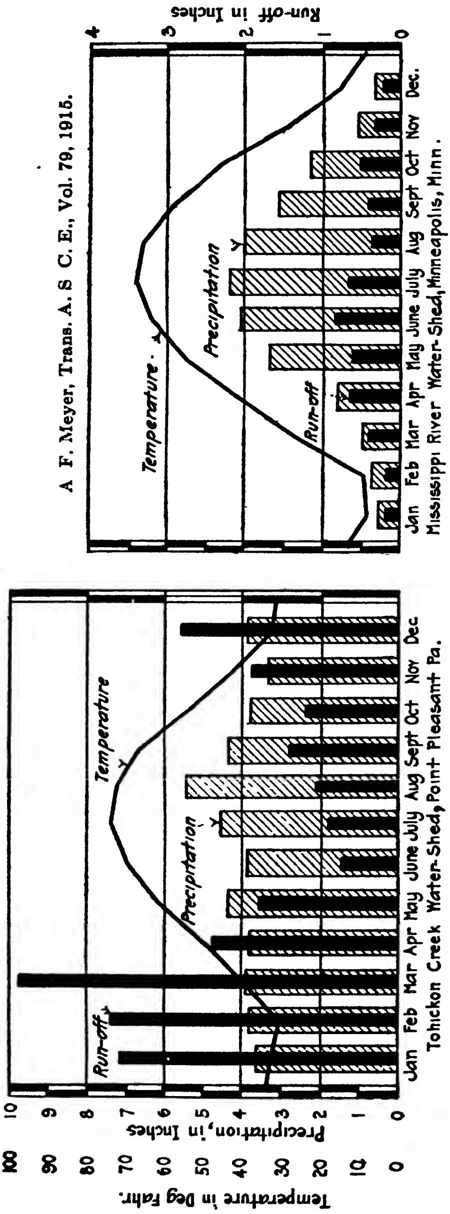

FLOW OF TYPICAL STREAMS OF NORTHEAST U.S.:
IN AN AVERAGE YEAR
FOR WATER POWER ESTIMATES
J.R. Freeman 1900
Mean line adopted is substantially mean of revised Croton records for 32 years and of Connecticut for 13 years giving greater weight to Croton.
Mean Values Adopted
Cu.ft. sec. per sq. mile
Driest Month 0.35
2nd " 0.45
3rd " 0.58
4th " 0.75
5th " 0.97
6th " 1.20
7th " 1.45
8th " 1.75
9th " 2.25
10th " 2.75
11th " 3.45
Wettest " 4.50
Sudbury-unusual evaporation
Unusual Storage on Merrimack
Very large water-shed with broad meadows and unusual ground storage
Croton 32 Yrs. 338. sq. mi.
20 sq. mi. Croton west branch. 3 years. 70-'71-'72
Connecticut 13 Years. 11,083 sq. mi.
23 Years '75 to '97 incl.
4,600 sq. mi. Merrimack
325 sq. mi. Concord 2 years 7 months '74-'75-'76
Mean Line
Driest
2
3
4
5
6
7
8
9
10
11
Wettest
Months in Order of Dryness
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Average run-off in cubic feet per second per square mile

FIG. 11.

allowance being made for the differing conditions. Although the average of a 40-yr. period may differ from that of a 60-yr. period, there is a strong probability that when the record has covered 80 or 100 yrs., a long enough period to complete more than one cycle of wet and dry years, the average will not be far from that for 40 yrs. (For rainfall data, see p. 12; for run-off data, see pp. 46 and 54.) Area of Croton Watershed is 360.4 sq. mi.*



A F. Meyer, Trans. A. S C. E., Vol. 79, 1915.

FIG. 12.—Type of diagram for showing relation of precipitation, run-off and temperature.

Run-off from Small Watersheds. C. C. Vermeule's 1894 Report on Water Supply of New Jersey shows that small watersheds have less run-off per sq. mi. than large ones. Minimum flow of seven streams, each with a catchment area of over 2000 sq. mi., was 0.182 c.f.s. per sq. mi.; twelve streams with areas between 200 and 2000 sq. mi. yielded 0.161; and twelve streams having areas less than 200 sq. mi. showed 0.094 c.f.s.

Pacific Coast Run-off. C. E. Grunsky, after study of precipitation and run-off on Pacific Coast, concluded that run-off "may be expressed in identical percentages of precipitation expressed in inches." For example, if the annual rainfall = 22 in., run-off = 22 per cent. of 22 = 4.8 in. This is borne out by half the tables on pp. 51 to 66.

Croton Flow in Dry Years. From 1868 to 1914 inclusive, the flow of the Croton river averaged 398.4 mgd. or 1,105,400 gals. per day per sq. mi. The following flows occurred in successive dry years: Driest calendar year on record to 1914, inclusive, 209 mgd.; 2 consecutive years, 266; 3 yrs., 287; 4 yrs., 291; 5 yrs., 299; 6 yrs., 317. Area = 360.4 sq. mi.

* For equivalents of the several run-off units used above, see p. 623.

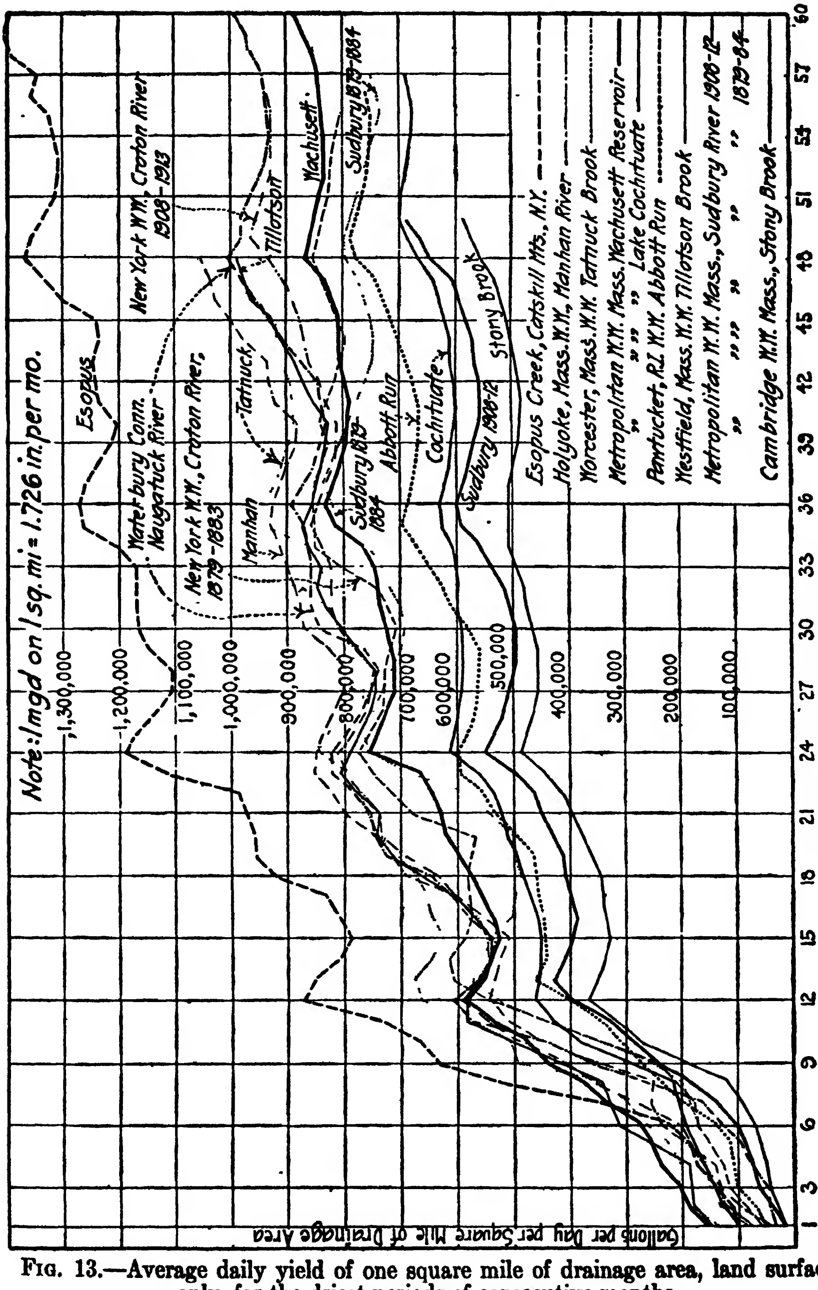


FIG. 13.—Average daily yield of one square mile of drainage area, land surface only, for the driest periods of consecutive months.

The severe drought, 1908–1913, is included. Curves show mean yields of drainage areas for numbers of consecutive months of minimum yield on bottom of diagram. N. E. W. W. Assn., Comm. on Yield of Drainage Areas, Jl. N. E. W. W. Ass'n., 1914.

Table 21. Run-off of Croton Watershed, June to Dec. each Year, 1868–1910, Compared with Total for Same Years; also with Rainfall

Year	Run-off, inches, June to Dec.	Per cent. of total run-off for year	Per cent of rainfall for same months	Year	Run-off, inches, June to Dec.	Per cent of total run-off for year	Per cent of rainfall for same months	Year	Run-off, inches, June to Dec.	Per cent. of total run-off for year	Per cent. of rainfall for same months
1868	18.2	55 1	59 1	1883	3 0	23 3	11 5	1897	12 3	48 2	35.5
1869	9 3	40 2	32 4	1884	7 8	33.3	26 1	1898	12 3	41 6	36.6
1870	3 1	16 3	13.8	1885	6 0	34 7	21 4	1899	5 1	22 5	20 4
1871	10 3	52 9	33 6	1886	4 0	20 6	15 8	1900	5 4	22 9	22 6
1872	8 6	50.9	30 0	1887	11 5	44 0	30 2	1901	17 3	47.2	44 6
1873	7.7	31.4	30 5	1888	15.7	45.1	43 8	1902	11 6	34 6	36 2
1874	5 2	20.8	22 7	1889	19.4	62.7	49.6	1903	17.3	49.6	43.3
1875	10.8	43.6	38 2	1890	11.1	43.5	34 9	1904	9 7	38.7	31.0
1876	*2 8	13 5	14 2	1891	3 9	16.2	15 8	1905	6 6	33 1	22 3
1877	8 2	41 6	24 8	1892	6 3	36.1	23 5	1906	6 4	31 5	23.5
1878	14 7	55 0	41 6	1893	9 2	32 1	32 3	1907	16 8	57 6	35.5
1879	6 0	31 0	20 3	1894	8 8	43.6	30 2	1908	2 9	14 6	15 2
1880	1 9	15.6	8 8	1895	3 5	22 4	13 9	1909	3 7	18 5	14 3
1881	4 6	24.5	18.7	1896	6 0	26 3	23 7	1910	2 9	15.3	14 1
1882	9 4	38 4	30 8								

Lowest month, Sept, 1881 — 0 03 in Evaporation exceeded rainfall * Lowest 7 months.

Table 22. Comparison of Run-off in Years of High and Low Precipitation.† (Calendar Years)

Drainage area	Period	Average precipitation, inches	Average run-off, inches	Per cent. of run-off
Wachusett (Mass.)	Highest 4 yrs	54.5	29 3	53.7
	Lowest 4 yrs.	38.7	17.1	44.2
	Difference	15.8	12.2	77.0
Sudbury (Mass.)	Highest 6 yrs	55.2	29.1	52.7
	Lowest 6 yrs	36.4	13 5	37.2
	Difference	18.8	15.6	82.9
Croton (N. Y.)	Highest 6 yrs	59.4	31.9	53 7
	Lowest 6 yrs	40.6	18.5	45.6
	Difference	18.8	13.4	71.3
Aver. Perkiomen, Neshaminy, and Tohickon (Penna.)	Highest 4 yrs	55.0	29.2	53.0
	Lowest 4 yrs.	42.3	19.5	46.0
	Difference	12.7	9.7	76.2
All sources	Highest 20 yrs	56.3	30.0	53.3
	Lowest 20 yrs	39.3	16.9	43.1
	Difference	17.0	13.1	77.0‡

† J. N. E. W. W. A., 1914. ‡ Obtained by dividing 13.1 by 17.0, and so for other differences.

It is the very high percentage of run-off from the difference in precipitation which accounts to a considerable extent for the difference in the run-off per square mile of different drainage areas in Table 22.

Niagara River Flow. Quantity flowing over Niagara Falls is 220,000 to 265,000 cu. ft. per sec. according to one authority. About 5 per cent. flows over the American Falls. Average flow is very uncertain, due to wind conditions on Lake Erie. According to gagings of Canadian government engineers at the time of a controversy over water to be taken for power, the flow varies from 300,000 to 700,000 cu. ft. per sec. Flow was estimated at 275,000 cu. ft. by John Bogart in 1890.

Table 23. Observed and Computed Physical Data for Fifteen Watersheds. (Calendar Years)

ADOLPH F. MEYER, Trans. A. S. C. E., Vol. 79, 1915, p. 1104

Reference No.	Name of watershed (river)	Years of record	Evaporation coefficient‡	Area, in square miles	Observed and computed physical data—mean annual, inches								Time of record
					Rainfall	Temperature	Evaporation	Transpiration	Total loss	Precipitation minus total loss	Computed run-off	Observed run-off	
1	Mississippi	17	1 20	19,500	27 3	41	14 4	7 7	22 1	5 2	5 23	5 31	1897–1913
2	Little Fork	5	1 10	1,720	23 9	37	11 2	6 9	18.1	5.8	5 80	. .	------
											*5 15	*5 13	
3	Minnesota	5	1.25	6,300	22 7	43	14.1	7.5	21.6	1 1	1 1	.	------
										*0 77	*0 77	*0 70	
4	Root	6	1.225	1,560	31.2	45	16 5	8 6	25.1	6.1	6 10		------
											*5 40	*5 20	
5	Ottertail	6	1.10	1,310	23.0	40	13 5	6 6	20.1	2.9	2 80		------
											†2 66	†2 59	
6	St Croix	11	1 05	5,930	30.0	41	13 1	7 0	20 1	9 9	9 90	9 60	1901–1912
7	Ohio	14	0 875	23,820	41 1	51	14 8	5 8	20 6	20 5	20 50	22 00	1891–1905
8	Tohickon Creek	24	0 90	102	48 9	51	16 7	7 0	23 7	25 2	25 2	26 10	1887–1911
9	James	7	0 925	6,230	42 1	54	16 3	7 0	23 3	18 8	18 90	18 00	1898–1905
10	Roanoke	9	0 90	390	42 6	57	16 9	7 0	23 9	18 7	18.60	17 70	------
11	Tombigbee	9	1 05	4,400	49 2	62	22 8	8 4	31 2	18 0	18 00	17 10	1900–1909
12	Colorado	10	1 20	37,000	26 9	66	17 7	8 1	25 8	1 1	1 06	0 74	1900–1910
13	Sacramento	9	0 85	10,400	32 2	52	8 5	2 4	10 9	21 3	21 3	20 40	1902–1911
14	Pit	6	1 10	2,950	14 8	48	6 9	3 0	9 9	4 9	4 9		1903–1908
										*3 87	*3 87	*3 92	1903–1908
15	McCloud	6	0 60	608	61 9	55	8 2	2 4	10 6	51 3	51 3	54 00	1902–1908

* Four years' records. † Five years' records. ‡ Defined on p. 31, last paragraph

1. At Minneapolis; elevations along river, 800 to 1475 ft., slope 1.5 ft. per mi.; some lake and swamp areas, but no allowance for loss therefrom except slight increase of evaporation coefficient; ⅓ under cultivation, remainder forests and brush.

2. At Little Fork, northern Minn.; elevation 1100 to 1400 ft., slope along river 2 ft. per mi.; flat topography, only few hills more than 50 to 75 ft. above plain; extensive swamps; clayey soil; generally densely wooded.

3. At Montevideo, western Minn.; slope of upper river 20 ft. per mi., lower river 0.6 ft. per mi.; flat open prairie, almost all under cultivation; large marshes with clayey soil near headwaters; water surfaces estimated 5 per cent. of watershed; transpiration limited by available moisture.

4. At Houston, southeastern Minn.; flat to gently undulating, streams in deep V-shaped valleys; mostly cultivated, woods along streams; soil is clayey loam, causing high evaporation losses; slope of river, 6 ft. per mi.

5. At Fergus Falls, western Minn.; dotted with lakes, water surfaces totaling 15 per cent. of watershed; lightly timbered, with much land cultivated; well-sustained stream flow; prominently rolling, morainic and knolly; heavily covered with drift.

6. At St. Croix Falls, Wis.; undulating, thickly covered with glacial drift; no marshes, numerous lakes, good ground storage; elevations along river 750 to 1000 ft.; slope of river 2.5 ft. per mi.

7. At Wheeling, W. Va.; northeastern portion rolling and hilly, remainder mountainous; shallow soil; slope along Allegheny river 6.5 ft. per mi.; heavy snow and thick ice common in Allegheny basin, but winters are less severe in upper Monongahela valley.

8. At Point Pleasant, southeastern Pa.; slopes steep; $\frac{3}{4}$ of area cultivated; $\frac{1}{4}$ wooded and untillable; slope of creek 20 ft. per mile.

9. At Cartersville, Va.; upper portion of watershed mountainous and wooded, elevations up to 4000 ft.; remainder is rolling plateau. (See also p. 60.)

10. At Roanoke, Va.; rugged with steep slopes along river; elevations up to 3000 ft.; soil scant; little land cultivated. (See also p. 62.)

11. At Columbus, Miss.; flat country; streams have gentle uniform slope; $\frac{1}{4}$ cultivated, remainder heavily forested; deep soil, heavy loam; humidity not much higher than in Northwest; average annual snowfall 4 in.

12. At Austin, Tex.; comparatively flat, but slopes pronounced; timber along streams and at higher elevations; soil deep and not sandy; humidity and wind differ but slightly from those in Northwest.

13 to 15. At Red Bluff, Cal.; about $\frac{1}{2}$ covered with timber, remainder mostly grass land; scant soil cover over lava; most of precipitation, especially at headwaters, is snow; elevations mostly 3000 to 5000 ft.; main portion of discharge comes from Pit river basin, which is relatively flat and largely pasture land.

Flood Flows. For list giving flood flows in cu. ft. per sec. (averaged over 24 hrs.), catchment area and duration of record, see paper by Weston E. Fuller, T. A. S. C. E., Vol. 77, 1914, p. 564. In 1901 report on Barge Canal, New York State, Emil Kuichling epitomizes formulas proposed for obtaining maximum flood flows.

Unusual Streamflow in the Southern States. Widespread floods in the Southern States, May and June, 1901, were reported to the U. S. Geological Survey* by E. W. Myers. Six days subsequent to May 17, 4.49 in. of rain fell in North Carolina; the normal fall for 6 days is about 0.84 in. The maximum fall for 24 hrs. for the eastern district was 3.33 in., 25 times the normal. Local records for 48 hrs., 7.95 in. (at Marion); for 6 days, 8.68 in. (at Chapel Hill). The Catawba river, near Rock Hill, S. C., gave a flood discharge of 150,800 cu. ft. per sec. from a drainage area of 2987 sq. mi. = 50 cu. ft. per sec. per sq. mi. The greatest previous flood was less than 100,000 cu. ft. per sec. Cane Creek, Mitchell Co., N. C., has 1650 ft. fall in 11 mi. The banks are steeply sloping, covered with thin soil. On account of the great fall, the ordinary flood hight rarely exceeded 6 ft.; 1901 flood rose to 12 ft. Boulders of 2 to 8 cu. yds., weighing from 4 to 16 tons, were carried 100 to 300 ft. Velocity, V, required to move bodies of mean diam., D, and specific gravity, g, immersed in water, was derived by Ganguillet and Kutter's formula, $V = 5.67\sqrt{Dg}$. If $g = 2.7$ (gneiss), and $D = 5$ ft., $V = 20.6$ ft. per sec. The area of watershed was 22 sq. mi.; the extreme low water discharge, 8 to 10 cu. ft. per sec. per sq. mi. Flood discharge was estimated to be 1340 cu. ft. per sec. per sq. mi. At Charleston, S. C., 5.0 in. of rain fell in 11 hrs., Sept. 11, 1912, of which 3.0 fell in 2 hrs. on Goose Creek Watershed (49.1 sq. mi.) (J. H. Gregory).

* E. N., Aug. 7, 1902, p. 103.

Table 24. Discharge and Sediment of Large Rivers*

River	Drainage area, sq. mi.	Mean annual discharge, sec. ft.	Sediment		
			Total annual, tons	Ratio, by weight to discharge	Depth over drainage area, in.
Potomac	11,043	20,160	5,557,000	1:3,575	0 00433
Mississippi	1,214,000	610,000	406,250,000	1:1,500	0 00288
Rio Grande	30,000	1,700	3,830,000	1:291	0 00110
Uruguay	150,000	150,000	14,783,000	1:10,000	0.00085
Rhone	34,800	65,850	36,000,000	1:1,775	0.01071
Po	27,100	62,200	67,000,000	1:900	0.01139
Danube	320,300	315,200	108,000,000	1:2,880	0.00354
Nile	1,100,000	113,000	54,000,000	1:2,050	0.00042
Irrawaddy	125,000	475,000	291,430,000	1:1,610	0.02005

Table 25. Quantities of Solids Carried in Suspension by Several American Rivers. In Parts per Million

River	Place	Max.	Min	Average		Median†	Authority	Remarks
				Quan.	Yrs			
Merrimac	Lawrence	1100–1500(a)	5	7 5	7	– – –	Clark	(a) A few days in 2 or 3 high floods in 20 yrs, mostly fine river silt, not within 7 yrs. 1908–1914 used in average.
Potomac	Great Falls.	3000(c)	6	119 (b)	8(a)	– – –	Hardy	(a) Jul. 1, 1906–Jun. 30, 1914 (b) Min. average for a year, 41, max. 208. (c) Lowest max. for a yr, 1500. (From daily turbidity readings, coef. of fineness is about one.)
Allegheny	Pittsburg.	3900(a)	1(a)	57	6	40	Drake	Turbidities are given. Coef of fineness about 0 8. (a) Infrequent Aver. susp. solids 1914 = 36.
Scioto	Columbus	2000	3	69	5(a)	20(b)	Hoover	Turbidities are given (a) 1909–1913; (b) for 1914
Ohio	Cincinnati.	5000(d)	5(e)	192 (c)	7(a)	60(b)	Ellms	Turbidities are given: (a) 1908–1914. (b) For 1914, average for that yr 120 (c) From daily determinations. (d) Has occurred a number of times. (e) Frequent.
Ohio	Louisville.	6000(b)	15(c)	235	4(a)	– – –	Leisen	Turbidities are given (a) 1910–1913 (b) Once in 1913; 5000 other years. (c) Monthly average 1911; max. monthly average, 1913, was 975.
Mississippi	St. Louis	6560	3	1400	7(a)	– – –	Wall	(a) May, 1905–Aug, 1912. Average turbidity, 1906–1913 was 1340; suspended solids, 1470.
Mississippi	New Orleans.	2400	55	600	6(a)	– – –	Earl	(a)1909–1914. Turbidities are given.

* C. C. Babb, E. N., Aug. 10, 1893, *q v.*, for authorities.
† By median is meant magnitude of item midway in list of all items in group considered, when these items are arranged in order of magnitude.

Turbidity Coefficient. Optical determinations of turbidity (see p. 788) are naturally compared with gravimetric determinations of suspended matter. Equal weights of suspended matter do not necessarily produce same turbidity; *e.g.*, silt or sand produce less than finely divided clay. Therefore, the ratio between silica turbidity determined optically and suspended matter determined gravimetrically is important as an index of the character of the suspended matter. Turbidity Coefficient = Suspended Matter ÷ Silica Turbidity. It varies with different waters, generally increasing with the size of the particles composing the suspended matter. (G. G. Earl, Rept. on Water Purification, New Orleans, 1903.)

Rainfall and Run-off. Explanation of Tables. The following tables were compiled from Water Supply & Irrigation Papers on Stream Flow (U. S. Geol. Survey) and from Monthly Weather Reviews. The second decimal is dropped from rainfall and run-off records, as explained on p. 3. "Rainfall year," Oct. 1 to Sept. 30, rather than calendar year, is used, so that the winter months may be kept together, as the larger per cent. of run-off occurs in the winter months, in the localities considered, excepting where melting snow on high mountains is important. Quantities of rainfall and run-off are in inches depth. Quantities under "Winter months," "Summer months," and "Yearly Means" are totals for those periods. All references to years is to rainfall year, beginning Oct. 1, *i.e.*, 1903–1906 is understood to be Oct. 1, 1903 to Sept. 30, 1906. Elevations refer to hight above sea-level and are intended merely as a guide in comparing watersheds. The lower elevation represents elevation where observation was made. The upper elevation represents the source, and not the highest point on the watershed. Stations are arranged geographically.

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.

Connecticut River above Orford, N. H.	Time groups / Description of data	Winter months, Dec.–Apr.: Precipitation	Winter: Run-off	Winter: Percentage of run-off	Summer months, June–Sept.: Precipitation	Summer: Run-off	Summer: Percentage of run-off	Yearly means, Oct. 1–Sept. 30: Precipitation	Yearly: Run-off	Yearly: Percentage of run-off
Descriptive		a	b		c	d		e	f	
Area, 3300 sq. mi. Steep slopes, mostly forested 20 sq. mi of water surface (estimate).	Means for 9 yrs. 1900–1909	12 8	11.1	87	15 3	4 4	29	36.3	21.9	60
	Max. (a) 1901–02 (b) 1901–02 (c) 1905 (d) 1902 (e) 1901–02 (f) 1901–02	17 0	14.8		19 3	7 9	.	41.8	27.0	
	Min (a) 1904–05 (b) 1903–04 (c) 1908 (d) 1909 (e) 1905–06 (f) 1903–04	8 9	6 8		11 8	2 6		32 2	16.0	

Descriptive	Description of monthly data (9 yrs.)			Oct	Nov.	Dec	Jan	Feb.	Mar.	Apr.	May	June	July	Aug	Sept.
Elevation, 380–6300 ft															
Mean temp., 1899–1910, at Chelsea, Vt. (10 mi west of gaging station). Jan., 24° F. July, 73° F.	Monthly means	Rainfall	Amount	3.0	2.1	2 8	2 3	2 1	2 8	2 9	3 2	3.9	4.0	3 6	3 9
		Run-off	Amount	1 3	1 2	1.4	1.0	0.8	2.8	5 0	3 9	1 4	1 0	0.9	0 8
	Monthly max	Rainfall	Year	1907	1900	1901	1909	1909	1902	1901	1901	1903	1905	1905	1907
			Amount	5 2	5 5	4 8	4 0	3 7	4 8	4 8	4 8	5 3	5.1	4.6	7.4
		Run-off	Year	1907	1907	1907	1906	1908	1903	1909	1907	1902	1907	1902	1905
			Amount	3.1	2.6	3 2	2 2	1 6	7 6	8.6	5 6	3 2	1 6	1.5	1.8
Run-off for driest 3 yrs., 1903–1906, 56 2 in. Aver. 18.7 in.	Monthly min	Rainfall	Year	1905	1904	1904	1907	1901	1909	1906	1903	1904	1909	1907	1908
			Amount	1.3	1.0	1 4	1.6	0 7	1.7	1.0	0 3	2.1	2.8	2.1	1.0
		Run-off	Year	1908	1908	1908	1904	{1904 1905	1906	1903	1903	{1903 1904	{1904 1908 1909	1909	1908
			Amount	0 3	0 4	0 4	0 3	0 3	0 8	3 6	1.2	1 0	0 5	0 3	0 2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Sudbury River	Time groups / Description of data	Winter months, Dec–Apr: Precipitation	Winter months, Dec–Apr: Run-off	Winter months, Dec–Apr: Percentage of run-off	Summer months, June–Sept.: Precipitation	Summer months, June–Sept.: Run-off	Summer months, June–Sept.: Percentage of run-off	Yearly means, Oct 1–Sept 30: Precipitation	Yearly means, Oct 1–Sept 30: Run-off	Yearly means, Oct 1–Sept 30: Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 75 sq mi Hilly, with steep slopes; large swamp areas 30 per cent. is forested Water surface = 5.20 sq mi	Means for 35 yrs, 1875–1910	20 2	15 3	76	13 9	2 1	15	45 4	21.5	47
	Max. (*a*)1890–91 (*b*)1890–91 (*c*)1889 (*d*)1889 (*e*)1890–91 (*f*)1890–91	27.9	24 8	. .	20.5	6.1	....	55.9	33.4	...
	Min. (*a*)1882–83 (*b*)1882–83 (*c*)1883 (*d*)1899 (*e*)1882–83 (*f*)1882–83	12.6	8 0	... .	7.3	0.3	... .	27.4	11.6	. .....

	Description of monthly data (35 years)			Oct.	Nov	Dec	Jan	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.
Elevation, 150–500 ft															
Mean Jan temp., 26° F	Monthly means	Rainfall	Amount	4 0	3.8	3 8	4 2	4 3	4 4	3 5	3 3	3.1	3.6	3.8	3.5
		Run-off	Amount	0.8	1 4	1 8	2 2	3 1	4 9	3.4	1.5	0 9	0.3	0.4	0.4
Mean July temp., 71° F (13 years).	Monthly max	Rainfall	Year	1895	1888	1901	1891	1900	1877	1904	1901	1903	1876	1898	1907
			Amount	10 7	7 2	9.7	7 0	9 1	8 4	8 9	7 2	9.2	9.1	8.2	8.8
		Run-off	Year	1890	1895	1878	1891	1886	1877	1901	1901	1903	1897	1889	1905
			Amount	3.9	4 8	5 5	5 2	8 3	8.3	7 2	5 1	3 4	1.1	2.5	2 2
Run-off for driest 3 yrs, 1907–10, 48.0 in, Aver 16 0 in	Monthly min	Rainfall	Year	1897	1908	1875, 1877	1876, 1901	1877	1910	1892	1903	1908	1885, 1888	1883	1877
			Amount	0.5	1 0	0.9	1 8	0 7	0 8	0.8	0 9	0.9	1.4	0.7	0.3
		Run-off	Year	1909	1908	1908	1883	1901	1889	1910	1910	1899	1900, 1908, 1909, 1910	1899, 1900, 1907, 1909, 1910	1908
			Amount	0 0	0 1	0 2	0.6	0 5	2 3	1.2	0.5	0.1	0.0	0.0	0.0

See also Rainfall Records, p. 9.

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Wachusett (south branch of Nashua River, at Clinton, Mass.)	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 118.2 sq. mi. Hilly, some broad valleys, gentle to steep slopes. 40 per cent. forested. Large quantity of sand and gravel. Water surface = 10 sq. mi.	Means for 13 yrs. 1897–1910	20.8	15.9	76	16.3	3.4	21	47.3	23.9	50
	Max. (*a*) 1901–02 (*b*) 1901–02 (*c*) 1903 (*d*) 1903 (*e*) 1897–98 (*f*) 1898–99	26.7	21.5		20.6	6.2	.	54.8	30.0	. .
	Min. (*a*) 1906–07 (*b*) 1906–07 (*c*) 1908 (*d*) 1900 (*e*) 1909–10 (*f*) 1909–10	13.8	10.5		12.6	2.0		37.3	17.6	.

	Description of monthly data (13 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 400–2000 ft. .	Monthly means	Rainfall	Amount	3.5	3.3	4.5	3.8	4.1	4.2	4.1	3.3	4.1	3.9	4.3	4.1
		Run-off	Amount	1.0	1.5	2.3	2.2	2.6	4.8	3.9	2.1	1.4	0.6	0.7	0.7
Mean Jan. temp., 24° F Mean July temp., 71° F (13 yrs.)	Monthly max.	Rainfall	Year	1898	1897	1901	1898	1900	1899	1901	1901	1903	1901	1898	1907
			Amount	7.2	7.6	9.4	6.6	8.7	6.8	9.6	7.0	10.4	5.7	10.6	9.5
		Run-off	Year	1898	1907	1901	1899	1900	1902	1901	1901	1903	1906	1898	1905
			Amount	2.6	4.4	5.6	3.6	7.0	6.9	8.6	4.7	3.7	1.3	2.3	2.1
Run-off for driest 3 yrs., 1904–07, 56.9 in. Aver 19.0 in.	Monthly min.	Rainfall	Year	1897	1902	1899	1901	1901	1910	1899	1905	1908	1910	1907	1908
			Amount	0.9	0.9	2.0	1.8	1.1	1.1	1.9	0.8	1.3	1.5	1.3	1.0
		Run-off	Year	1909	1908	1899	1901	1901	1907	1910	1905	1908	1910	1907	1908
			Amount	0.2	0.2	0.6	0.9	0.6	2.9	1.8	0.8	0.7	0.1	0.2	0.2

During the dry period, 1907–1913, low run-offs occurred as follows Oct., 1907–Sept. 1908, 26.6 in., 1908–1909, 18.8, 1909–1910, 17.6; 1910–1911, 10.6; 1911–1912, 20.9, 1912–1913, 16.6. Driest 3 yrs., Oct., 1908–Sept., 1911, 57.0 in. Average, 19.0. The average, 18.5 in. for these 6 yrs., was little more than ¾ of the average, 24.7 in., for preceding 10 yrs. See also Rainfall Records, p. 10.

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Croton River at New Croton Dam, New York	Time groups / Description of data	Winter months, Dec.-Apr. Precipitation	Run-off	Percentage of run-off	Summer months, June-Sept. Precipitation	Run-off	Percentage of run-off	Yearly means, Oct 1-Sept. 30 Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 360.4 sq mi Rolling, largely agricultural and grazing; water surface = 5.36 per cent. total watershed.	Means for 43 yrs, 1868–1911	20.1	15.2	76	17.3	3.5	20	48.9	23.3	47
	Max. (*a*)1908–09 (*b*)1901–02 (*c*)1887 (*d*)1901 (*e*)1900–01 (*f*)1902–03	26.9	24.1		26.4	8.9		61.4	34.0	. . . .
	Min. (*a*)1871–72 (*b*)1870–71 (*c*)1881 (*d*)1880 (*e*)1876–77 (*f*)1870–71	10.9	7.9		10.5	0.9	.	37.1	13.7	

Descriptive	Description of monthly data (43 yrs.)			Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept
Elevation, 200–1400 ft	Monthly means	Rainfall	Amount	3.9	3.8	3.9	4.2	4.2	4.2	3.5	3.7	3.6	4.7	5.0	4.0
		Run-off	Amount	1.2	1.6	2.1	2.7	2.9	4.2	3.2	1.9	1.0	0.7	0.9	0.8
Mean Jan temp., 25° F. Mean July temp., 77° F (1912).	Monthly max	Rainfall	Year	1869	1889	1901	1891	1900	1877	1901	1893	1903	1887	1898	1882
			Amount	9.5	8.5	8.8	9.1	7.7	8.1	8.2	7.9	11.3	11.2	11.5	14.6
		Run-off	Year	1903	1907	1878	1874	1900	1902	1901	1893	1903	1897	1875	1882
			Amount	4.8	5.0	7.1	8.3	6.4	9.9	7.5	5.5	3.2	2.6	5.1	3.4
Run-off for driest 3 yrs., 1908–1911, 52.5 in. Aver. 17.5 in.	Monthly min.	Rainfall	Year	1879	1902	1892	1896	1901	1910	1892	1887	1873	1873	1899	1881, 1884
			Amount	0.7	0.9	1.0	1.1	0.8	0.7	1.1	0.3	0.7	2.2	0.6	0.8
		Run-off	Year	1910	1908	1876, 1880	1871, 1875	1901	1872	1882	1896	1880	1891, 1909	1880, 1881, 1882, 1900, 1907	1881, 1910
			Amount	0.0	0.2	0.4	0.5	0.5	1.5	1.2	0.5	0.3	0.1	0.1	0.0

See also Rainfall Records, p 12, and Croton flow in dry years, p 44

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Housatonic River, above Gaylordsville, Conn.	Time groups	Winter months, Dec–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 1020 sq. mi. Extensive cultivation; sparsely timbered hills.	Means for 9 yrs., 1900–09	18 3	15 5	85	18 4	5 5	30	47 1	26 6	56
	Max. (*a*) 1908–09 (*b*) 1901–02 (*c*) 1903 (*d*) 1903 (*e*) 1907–08 (*f*) 1902–03	25.2	21.6		23 3	10 6		52 9	36.9	... ...
	Min. (*a*) 1906–07 (*b*) 1906–07 (*c*) 1909 (*d*) 1908 (*e*) 1904–05 (*f*) 1908–09	13.0	9 9	. ..	13.0	2 8	..	39.8	19.4	......

Descriptive	Description of monthly data (9 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 230 to 1900 ft.															
Mean temp., 1898–1910, at Carmel, N. Y. (18 mi. southwest of gaging station). Jan., 25° F. July, 72° F.	Monthly means	Rainfall	Amount	4.0	2.5	4 1	3.3	3.3	3.8	3.9	3 7	4.5	4.5	4.7	4.7
		Run-off	Amount	1.8	1.6	2 3	2.3	1.9	4 7	4 3	2 5	2 0	1.3	1.2	1 2
	Monthly max.	Rainfall	Year	1907	1907	1901	1905	1909	1901	1909	1901	1903	1902	1901	1907
			Amount	6.8	5.5	6.8	4.9	6 6	5 2	6 2	6 2	10.4	7.2	7.3	10.6
		Run-off	Year	1907	1907	1902	{1902 1908	1903	1902	1901	1901	1903	1903	1903	1905
			Amount	4 2	5 4	4 1	3.2	3 7	8 8	6 4	4.5	4.3	2.5	2 2	2.2
Run-off for driest 3 yrs., 1904–07, 67.4 in. Aver., 22.5.	Monthly min.	Rainfall	Year	1908	1902	1900	1901	1901	1907	1903	1903	1908	1909	1907	1908
			Amount	2.1	0.9	2.7	1 6	0.8	1.3	2.2	1.1	1.7	2 0	1.8	1.4
		Run-off	Year	{1900 1908	1908	1908	1901	1901	1907	1907	1903	1905	{1905 1909	1907	1908
			Amount	0.4	0 3	0.5	1.0	0.5	2.7	2.6	1.1	0.9	0.6	0.6	0.3

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

West branch of Susquehanna River above Williamsport, Pa.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 5640 sq mi. Narrow valley, high wooded banks	Means for 14 yrs., 1895–1909	16 0	14 6	91	15 0	3 5	23	39 8	22 2	56
	Max. (*a*)1907–08 (*b*)1902–03 (*c*)1902 (*d*)1902 (*e*)1907–08 (*f*)1901–02, 1902–03	19 7	19 8		19 7	6 1		44 2	27.6	. .
	Min. (*a*)1904–05 (*b*)1895–96 (*c*)1909 (*d*)1899 (*e*)1899–1900 (*f*)1895–96	12 5	11 0	.	10 1	1 6		33 0	16 6	. .

	Description of monthly data (14 yrs.)			Oct	Nov.	Dec	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug	Sept.
Elevation, 530–2000 ft.															
Mean temp., 1898–1910, at Williamsport. Jan., 25° F. July, 74° F.	Monthly mean	Rainfall	Amount	2 9	2 5	3.3	2 7	2 8	4.1	3.2	3 5	4.1	4.3	3.8	2.8
		Run-off	Amount	0.8	1.1	1.8	2 2	2 0	5 2	3 4	2.1	1.2	1.1	0.7	0.4
	Monthly max.	Rainfall	Year	1898	1897	1901	1907	1908	1898	1909	1908	1903, 1907	1902	1901	1907
			Amount	6.2	4.9	5.5	4.6	4 7	5 2	6 7	8.0	6.0	7.6	6.6	6.1
		Run-off	Year	1896	1900	1901	1907	1903	1902	1901	1908	1901	1902	1901	1901, 1903
			Amount	2 7	1 8	4 1	4 1	4 5	8 1	5 4	5.1	2 4	4.1	1.4	1.2
Run-off for driest 3 yrs., 1904–07, 58.8 in. Aver. 19.6 in	Monthly min.	Rainfall	Year	1901	1904	1896	1896	1905, 1906	1909	1900	1903	1908	1909	1907	1900, 1908
			Amount	0 9	0.5	1 2	1 5	1.0	2.6	1.3	1 7	2.1	1.7	1.4	1.0
		Run-off	Year	1895, 1899, 1908	1909	1904, 1908	1897	1901, 1905	1906	1907	1903	1899	1898, 1899, 1900, 1908, 1909	1907, 1909	1908, 1909
			Amount	0.2	0.2	0.3	1.0	0 6	1.6	1 8	0 6	0 5	0 4	0.2	0.1

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Susquehanna River above Wilkes-Barre, Pa.	Time groups	Winter months, Dec.–Apr.			Summer months, June-Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 9810 sq. mi. Rolling, broken country in N. Y., mountains in Pa.	Means for 10 yrs., 1899–1909	13.9	14.8	107	15.4	2.8	18	38.4	21.3	55
	Max (*a*)1901–02, 1902–03 (*b*)1901–02 (*c*)1902 (*d*)1903 (*e*)1901–02 (*f*)1902–03	16.7	20.8	. . .	21.0	4.8	....	44.1	27.2	....
	Min (*a*)1904–05 (*b*)1906–07 (*c*)1908 (*d*)1900, 1908 (*e*)1908–09 (*f*)1899–1900	12.3	10.7	....	9.8	1.1	....	30.4	15.2	....

	Description of monthly data (10 yrs.)			Oct	Nov	Dec	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, source 1193 ft. At Wilkes-Barre 525 ft.															
Mean temp., 1898–1910, at Wilkes-Barre. Jan., 23° F. July, 74° F.	Monthly mean	Rainfall	Amount	3.4	2.3	3.2	2.6	2.4	3.4	2.9	3.0	4.2	4.4	3.7	3.0
		Run-off	Amount	1.0	1.0	2.3	2.6	2.3	4.5	3.1	1.6	1.0	0.8	0.5	0.5
	Monthly max.	Rainfall	Year	1903	1900	1901	1904, 1907	1909	1903	1901	1901	1903	1902	1903	1907
			Amount	6.0	4.7	5.6	3.4	3.9	4.8	4.7	5.4	6.4	7.9	6.5	6.5
		Run-off	Year	1903	1907	1901	1907	1904	1902	1901	1909	1901	1902	1903	1905
			Amount	3.2	2.1	4.9	3.5	3.9	7.8	4.5	3.3	1.8	3.4	1.5	1.4
Run-off for driest 3 yrs., 1904–1907, 52.1 in. Aver 17.4 in.	Monthly min.	Rainfall	Year	1901	1902	1904, 1908	1901, 1906	1901, 1905	1907	1900	1903	1908	1909	1907	1908
			Amount	1.7	1.1	2.2	1.7	1.2	2.5	1.5	1.1	2.4	2.4	1.2	1.3
		Run-off	Year	1899, 1900	1908	1908	1908, 1909	1905, 1907	1906	1907	1903	1900	1909	1902	1908, 1909
			Amount	0.1	0.2	0.2	1.7	0.6	2.2	2.0	0.4	0.4	0.2	0.1	0.1

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Susquehanna River above Harrisburg, Pa.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area 28,030 sq. mi. Narrow valley, high wooded banks in Pa., rolling, broken country in N. Y.	Means for 18 yrs., 1891–1909	14.6	13.4	92	14.8	3.3	22	39.0	20.8	53
	Max. (*a*)1902–03 (*b*)1902–03 (*c*)1902 (*d*)1901 (*e*)1891–92 (*f*)1902–03	18.8	19.2	......	20.3	5.8	....	45.2	28.0	....
	Min. (*a*)1896–97 (*b*)1896–97 (*c*)1908 (*d*)1895, 1900 (*e*) and (*f*)1899–1900	11.4	10.8	....	10.4	1.4	....	31.6	16.3	....

	Description of monthly data (18 yrs.)			Oct	Nov.	Dec.	Jan.	Feb	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 300–1200 ft.															
Mean temp., 1898–1910, at Harrisburg. Jan., 30° F. July, 75° F.	Monthly means	Rainfall	Amount	3.1	2.5	3.0	2.7	2.7	3.3	2.9	3.9	3.9	4.0	3.8	3.1
		Run-off	Amount	0.9	1.1	1.8	2.0	2.0	4.3	3.3	2.2	1.2	0.8	0.7	0.6
	Monthly max	Rainfall	Year	1898	1897, 1900	1901	1892	1893	1903	1909	1894	1903	1902	1903	1907
			Amount	5.7	4.4	5.6	4.4	4.6	4.6	4.7	7.7	6.4	7.2	6.5	6.6
		Run-off	Year	1903	1894	1901	1892	1903	1902	1893, 1901	1894	1892	1902	1901	1903
			Amount	2.2	2.2	3.5	3.8	4.0	7.5	4.8	4.5	3.0	3.2	1.6	1.4
Run-off for driest 3 yrs., 1904–1907, 54.9 in. Aver. 18.3 in.	Monthly min.	Rainfall	Year	1892	1904	1896	1897	1901	1894	1896	1903	1908	1909	1907	1908
			Amount	1.0	0.9	1.0	1.8	0.9	1.2	1.3	1.3	2.2	2.2	1.6	1.3
		Run-off	Year	1895, 1899, 1900, 1908	1895, 1908	1908	1893, 1901	1905	1906	1907	1896	1895, 1899	1900	1895, 1900, 1909	1895, 1896, 1900, 1908, 1909
			Amount	0.2	0.3	0.3	0.7	0.5	2.1	1.8	0.6	0.5	0.3	0.2	0.2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

James River above Buchanan, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 2060 sq. mi. Open country.	Means for 14 yrs. 1895–1909	10.8	10.9	65	16.3	3.6	22	42.3	17.9	42
	Max. (*a*)1902–03 (*b*)1901–02 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1900–01	22.3	18.0		23.2	8.3	.	49.8	26.3	......
	Min. (*a*)1903–04 (*b*)1904–05 (*c*)1897 (*d*)1899 (*e*)1903–04 (*f*)1903–04	10.3	5.4		10.3	1.3	.....	30.4	11.4	....

	Description of monthly data (14 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 830–2500 ft															
Mean temp., 1898–1910, at Clifton Forge, Va (20 mi northwest of gaging station). Jan., 33° F. July, 75° F.	Monthly means	Rainfall	Amount	3.0	2.4	3.4	3.0	3.2	3.8	2.9	4.1	4.9	4.3	3.8	3.3
		Run-off	Amount	0.8	0.8	1.4	1.8	2.2	3.2	2.2	1.9	1.4	0.9	0.8	0.6
	Monthly max	Rainfall	Year	1898	1896	1901	1903	1897	1903	1901	1901, 1905	1903	1905	1901	1896
			Amount	8.1	5.2	7.6	4.6	5.8	5.7	6.5	6.3	7.6	8.5	8.7	6.2
		Run-off	Year	1906	1900	1901	1908	1897	1899	1901	1901	1907	1905	1901	1907
			Amount	4.1	2.5	4.8	3.7	5.3	5.7	5.0	3.6	3.9	3.0	2.7	1.2
Run-off for driest 3 yrs., 1903–1906, 37.7 in. Aver. 12.6 in	Monthly min	Rainfall	Year	1896, 1904	1899	1896	1897	1906	1904, 1905	1896, 1897, 1902	1903	1897	1902	1902	1897
			Amount	0.5	0.7	0.3	1.8	0.4	2.4	1.6	1.2	3.3	2.3	1.6	1.1
		Run-off	Year	1895, 1897, 1899, 1904	1897, 1899, 1904, 1905	1903, 1904	1897	1905	1898	1905	1896	1898, 1902	1899	1899, 1900	1897, 1902, 1904
			Amount	0.2	0.2	0.3	0.5	0.5	1.5	0.9	0.6	0.5	0.2	0.2	0.2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

James River above Cartersville, Va.	Time groups / Description of data	Winter months, Dec -Apr: Precipitation	Winter months, Dec -Apr: Run-off	Winter months, Dec -Apr: Percentage of run-off	Summer months, June–Sept.: Precipitation	Summer months, June–Sept.: Run-off	Summer months, June–Sept.: Percentage of run-off	Yearly means, Oct 1–Sept. 30: Precipitation	Yearly means, Oct 1–Sept. 30: Run-off	Yearly means, Oct 1–Sept. 30: Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 6230 sq. mi. Open country, rolling.	Means for 10 yrs, 1898–1908	16 8	10 7	64	17 6	4.4	24	43.8	18.6	42
	Max. (*a*)1902–03 (*b*)1902–03 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1902–03	21.6	15.8		24.8	8.7		54.8	24.7	....
	Min. (*a*)1903–04 (*b*)1903–04 (*c*)1902 (*d*)1902 (*e*)1903–04 (*f*)1903–04	10 4	5.3	.	12.8	2.0		30.6	10.5	......

Descriptive	Description of monthly data (10 yrs)			Oct	Nov.	Dec.	Jan.	Feb.	Mar.	Apr	May	June	July	Aug.	Sept.
Elevation, 265–4000 ft. ..															
Mean temp., 1898–1910, at Columbia, Va. (5 mi. northwest of gaging station). Jan., 37° F. July, 76° F.	Monthly means	Rainfall	Amount	3 6	2.1	3.6	3.2	3 1	3 8	3.1	3.7	5.2	4.0	5.0	3.4
		Run-off	Amount	1.1	0 9	1.6	2.0	2.0	2 9	2.2	1 6	1.6	0.9	1.1	0.8
	Monthly max	Rainfall	Year	1898	1907	1901	1903	1903	1899	1901	1901	1903	1905	1901	1907
			Amount	8 4	5 1	7 2	4.4	5.1	6.4	6.9	6.7	7.7	7.5	10.2	5.9
		Run-off	Year	1906	1906	1901	1908	1899	1899	1901	1901	1907	1905	1901	1907
			Amount	3.8	1 6	3.3	3.1	3.7	5.3	4.4	3.4	3.4	2.5	3.1	1.3
Run-off for driest 3 yrs, 1903–1906, 39. 8 in. Aver., 13.3 in.	Monthly min	Rainfall	Year	1904	1905	1903	1907	1901	1905	1899	1903	1899	1902	1900	1904
			Amount	0.5	0.8	1 4	1.7	0.6	2.3	1.7	1.8	3.6	2.4	1.5	2.0
		Run-off	Year	1904	1904	{1903 1904	1904	1901	1901	1905	{1900 1906	1899	{1899 1902	1900	{1902 1904
			Amount	0.2	0 3	0 5	0.7	0.6	1.5	1.0	0.9	0.7	0.4	0.3	0.3

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

North (of James) River above Glasgow, Va.	Time groups / Description of data	Winter months, Dec.–Apr. Precipitation	Winter Run-off	Winter Percentage of run-off	Summer months, June–Sept Precipitation	Summer Run-off	Summer Percentage of run-off	Yearly means, Oct. 1–Sept. 30 Precipitation	Yearly Run-off	Yearly Percentage of run-off
Descriptive		a	b		c	d		e	f	
Area, 830 sq. mi. Steep slopes, some swamps.	Means for 10 yrs. 1895–1905	15 9	9 6	60	15 9	3 6	23	40 8	16 0	39
	Max. (a)1902–03 (b)1901–02 (c)1896 (d)1901 (e)1900–01 (f)1900–01	21 8	16 1		22.4	7 7		51 5	21 3	.
	Min (a)1903–04 (b)1904–05 (c)1902 (d)1899 (e)1903–04 (f)1899–1900	11 4	5 4	.	10 0	1.1		32 5	12 2	.. .

	Description of monthly data (10 yrs)			Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept.
Elevation, 1100–2000 ft															
Mean temp, 1898–1910, at Lexington, Va. (8 mi. north of gaging station). Jan., 35° F. July, 73° F.	Monthly means	Rainfall	Amount	2 6	2 4	2 9	2 9	3 6	3 9	2 7	4 0	4 8	4.1	3 8	3 2
		Run-off	Amount	0 7	0 6	1 2	1 4	2 3	2 8	1 8	1 5	1 1	1.0	0 8	0 6
	Monthly max.	Rainfall	Year	1898	1896	1901	1902	1897 1903	1899	1901	1901	1903	1896 1905	1901	1896
			Amount	8 7	5 3	7 6	3 2	5 5	6.4	7 1	6 2	8 7	6 2	7 5	6 7
		Run-off	Year	1898	1896	1901	1903	1897	1899	1901	1897 1901	1901	1905	1901	1896
			Amount	2 5	1 7	4 5	2 8	4 0	4 4	4 2	2.8	3 0	2 8	2 5	2.1
Run-off for driest 3 yrs, 1897–1900, 42.4 in. Aver 14.1 in.	Monthly min	Rainfall	Year	1896	1899	1896	1897	1901	1905	1896	1903	1898	1902	1900	1897
			Amount	0 3	0 7	0 2	1 8	0 5	2 7	1 2	1 3	2 7	2 2	1 5	0 7
		Run-off	Year	1899 1904	1904	1899 1904	1897	1901	1898	1905	1902	1899	1899	1902	1897 1902 1904
			Amount	0 2	0 2	0 3	0 6	0 4	1 3	0 8	0 6	0 3	0 2	0 2	0 2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Roanoke River above Roanoke, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 390 sq. mi. Coastal plain, heavily timbered.	Mean for 9 yrs. 1896–1905*	16.5	9 8	59	16.8	4 4	26	42.7	17.7	41
No lakes. Swampy.	Max. (*a*)1902–03 (*b*)1898–99 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1900–01	22 3	19 8		26.6	12.1	. . . .	58.3	29.7	. . .
	Min. (*a*)1903–04 (*b*)1903–04 (*c*)1902 (*d*)1902 (*e*)1901–02 (*f*)1903–04	10 0	3 2		10 4	1.4	.	34.5	8.9	. . .

Descriptive	Description of monthly data (9 yrs.)			Oct	Nov	Dec.	Jan	Feb	Mar	Apr.	May	June	July	Aug.	Sept.
Elevation, 980–3000 ft..															
Mean temp., 1898–1908, at Roanoke. Jan., 36° F. July, 76° F.	Monthly mean	Rainfall	Amount	2 7	2 5	2 9	2 8	3 9	4.0	2 8	4 2	4.8	4 9	3.8	3 3
		Run-off	Amount	0 9	0 8	1 3	1.4	2 3	2 9	1 9	1 8	1 1	1.2	1.3	0.8
	Monthly max.	Rainfall	Year	1898	1902	1901	1903	1897	1899, 1903	1910	1901	1904	1905	1901	1900
			Amount	6 3	4 6	7 4	4 1	7 1	6 5	6 5	7.5	8 1	11 6	10 7	5.2
		Run-off	Year	1898	1900	1898	1899	1899	1899	1901	1901	1901	1905	1901	1903
			Amount	3.1	1 7	2.0	3 3	5.6	7.5	4 9	4.4	2 5	3.5	5.7	1.5
Run-off for driest 3 yrs., 1902–05, 44 4 in. Aver. 14.8 in	Monthly min	Rainfall	Year	1904	1901	1896	1897	1898	1905	1897	1903	1897	1900	1902	1904
			Amount	0 1	1 0	0 5	1 6	0 6	2 2	1 7	0 9	1 9	3 1	1.0	1.2
		Run-off	Year	1899, 1904	1897	1897, 1903, 1904	1897	1904	1898	1904	1902	1902	1899, 1902	1899, 1900, 1902	1902
			Amount	0 3	0 2	0 4	0 2	0 6	0 9	0 6	0 8	0 5	0 4	0.3	0.2

* Subsequent records are fragmentary.

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Roanoke River above Randolph, Va.	Time groups / Description of data	Winter months, Dec.–Apr.: Precipitation	Winter months: Run-off	Winter months: Percentage of run-off	Summer months, June–Sept.: Precipitation	Summer months: Run-off	Summer months: Percentage of run-off	Yearly means Oct. 1–Sept. 30: Precipitation	Yearly means: Run-off	Yearly means: Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 3080 sq. mi.	Means for 5 yrs. 1900–1905*	17 5	9 5	54	17.4	5.6	32	44 0	18 7	24
	Max. (*a*) 1902–03 (*b*) 1901–02 (*c*) 1901 (*d*) 1901 (*e*) 1900–01 (*f*) 1900–01	24.6	14.0	. .	24.1	10.3	.... ..	54.0	25 2	. ... .
	Min. (*a*) 1903–04 (*b*) 1903–04 (*c*) 1902 (*d*) 1902 (*e*) 1903–04 (*f*) 1903–04	10 2	4.6	. ...	11.8	3 6	. ..	34.0	11.0	.. .

Descriptive	Description of monthly data (5 yrs.)			Oct	Nov.	Dec.	Jan	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept
Elevation, approx. 400–3000 ft.															
Mean temp., 1902–1910, at Saxe, Va. (2 mi. northeast of gaging station). Jan., 38° F. July, 76° F.	Monthly means	Rainfall	Amount	2 6	2.2	3 9	3 1	3.5	3.6	3.3	4.0	4.5	4.9	5.1	2.8
		Run-off	Amount	1 0	0 8	1.7	1.6	2.0	2 3	1 9	1 7	1.3	1.4	1.8	1.0
	Monthly max	Rainfall	Year	1902	1900	1901	1903	1903	1903	1901	1905	1904	1905	1901	1901
			Amount	5.5	3.2	7.8	4.4	5 6	6 5	6 0	6 4	6.0	8.7	11.2	3.3
		Run-off	Year	1902	{1900 1901	1901	1903	1902	1903	1901	1901	1901	1905	1901	1901
			Amount	1 8	1 1	3.6	2.3	3.5	4 1	3 5	3.2	1.7	2 4	4.9	1.4
Run-off for driest 3 yrs., 1902–1905, 47 0 in. Aver. 15.7 in.	Monthly min.	Rainfall	Year	1904	1901	1903	1904	1901	1905	1904	1903	1905	1902	1902	1904
			Amount	0 6	1.2	1.8	2 2	0.9	2 3	1 5	1 9	2 8	2 7	2.4	1.9
		Run-off	Year	1904	1904	{1903 1904	1904	1901	1904	1904	1904	1905	{1902 1904	1902	1904
			Amount	0 3	0.3	0 7	0 8	1 0	1.0	0.8	1 1	1 0	0.8	0.8	0.6

* Rainfall records discontinued Aug 12, 1906.

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Shenandoah River above Millville, W. Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 3000 sq. mi. Broad valley and overflowed banks.	Means for 13 yrs., 1895–1908	14 1	7 7	55	16.4	3.4	21	39.1	14.1	36
	Max. (*a*)1901–02 (*b*)1901–02 (*c*)1896 (*d*)1901 (*e*)1900–01 (*f*)1901–02	19.1	16 0		22.3	7.1	.	48.1	19.8	. .
	Min. (*a*)1903–04 (*b*)1903–04 (*c*)1897 (*d*)1902 (*e*)1903–04 (*f*)1903–04	8.7	3.6		9.7	1 6		30.5	7.9	..

Descriptive	Description of monthly data (13 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 230–1380 ft															
Mean temp., 1898–1910, at Martinsburg, W. Va. (15 mi. northwest of gaging station) Jan., 31° F. July, 75° F.	Monthly means	Rainfall	Amount	2 6	2 1	2.8	2.6	2 9	3.4	2.5	3 8	5.1	4.2	4.0	3.2
		Run-off	Amount	0 9	0 6	1.1	1.4	1 5	2 0	1.7	1.4	1.3	0.8	0.9	0.5
	Monthly max.	Rainfall	Year	1898	1896	1901	1903	1897	1899	1901	1908	1903	1896	1898	1896
			Amount	7 1	4 2	6 1	4 1	6 3	5 1	6 2	6 5	7 6	6 2	7.7	7.2
		Run-off	Year	1906	1906	1901	1908	1897	1902	1901	1901	1901	1901	1898	1907
			Amount	3 2	1 1	3 1	2.9	3 9	5 3	4 8	3.4	3.1	1.7	3.2	1.0
Run-off for driest 3 yrs., 1903–1906, 27.6 in. Aver., 9 2 in.	Monthly min.	Rainfall	Year	1896	1899, 1903	1896	1897	1901	1908	1896, 1899	1907	1899	1902	1897	1897
			Amount	0 5	0 8	0 2	1 4	0.3	1.8	1 2	1.8	2.1	2.2	1.4	1.0
		Run-off	Year	1895, 1904, 1905	1895, 1904, 1905	1895, 1904	1897, 1904	1901	1898	1905	1905	1902	1899	1900	1897, 1904
			Amount	0 2	0 2	0 3	0.5	0.5	0.9	0 7	0.5	0.5	0.3	0.3	0.2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Potomac River above Point of Rocks, Md.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 9650 sq. mi. Steep, cultivated slopes; no lakes.	Means for 14 yrs. 1895–1909	14.4	9.1	63	14.9	2.4	16	37.4	14.4	38
	Max. (*a*)1901–02 (*b*)1901–02 (*c*)1903 (*d*)1901 (*e*)1902–03 (*f*)1901–02	19.9	18.5	.	20.3	5.4	. ..	44.8	21.5	..
	Min. (*a*)1903–04 (*b*) {1895–96, 1908–09} (*c*)1902 (*d*)1902 (*e*)1903–04 (*f*)1895–96	8.6	5.2	.	10.4	1.3	. .	29.4	8.2	.

Descriptive	Description of monthly data (14 yrs.)			Oct	Nov	Dec	Jan.	Feb	Mar.	Apr.	May	June	July	Aug	Sept.
Elevation, 300–1200 ft															
Mean temp., 1896–1904, at Sharpsburg, Md. (20 mi northwest of gaging station). Jan., 31° F. July, 76° F.	Monthly means	Rainfall	Amount	2.3	2.2	2.8	2.7	2.8	3.4	2.8	3.9	4.3	3.9	4.0	2.6
		Run-off	Amount	0.6	0.5	1.0	1.6	1.8	2.7	1.9	1.4	1.1	0.7	0.7	0.4
	Monthly max	Rainfall	Year	1898	1897	1901	1907	1897	{1898, 1902}	1901	1908	1903	1905	1906	1896
			Amount	6.4	4.1	5.7	4.1	5.9	4.4	6.0	7.8	6.6	6.6	7.7	6.1
		Run-off	Year	1906	1898	1901	1907	1897	1902	1901	1908	1907	1903	1898	1906
			Amount	2.0	1.0	3.1	3.2	4.6	6.5	4.6	3.8	2.7	1.5	2.7	0.9
Run-off for driest 3 yrs., 1903–1906, 30.5 in. Aver 10.2 in.	Monthly min	Rainfall	Year	1901	1904	1896	1897	1901	{1904, 1909}	1900	1902	1898	1901	1896	1898
			Amount	0.6	0.8	0.7	1.6	0.5	2.1	1.3	2.0	1.8	1.2	1.7	1.3
		Run-off	Year	{1895, 1904}	{1895, 1897, 1903, 1904}	1905	1897	1901	1909	1905	1896	1902	{1898, 1899, 1909}	{1900, 1909}	1896, 1900, 1902, 1904, 1909
			Amount	0.1	0.2	0.1	0.5	0.4	1.0	0.8	0.3	0.4	0.3	0.2	0.2

Table 26. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Concluded*)

Appomattox River above Mattoax, Va.	Time groups / Description of data	Winter months, Dec.–Apr. Precipitation	Winter Run-off	Winter Percentage of run-off	Summer months, June–Sept. Precipitation	Summer Run-off	Summer Percentage of run-off	Yearly means, Oct. 1–Sept. 30 Precipitation	Yearly Run-off	Yearly Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 745 sq mi	Means for 5 yrs, 1900–05	16.9	9.9	59	17.2	3.9	23	43.0	16.5	38
	Max. (*a*) 1902–03 (*b*) 1902–03 (*c*) 1901 (*d*) 1901 (*e*) 1900–01 (*f*) 1902–03	22.9	16.6	...	24.2	6.3		53.0	25.2	..
	Min. (*a*) 1903–04 (*b*) 1900–01 (*c*) 1902 (*d*) 1902 (*e*) 1903–04 (*f*) 1903–04	9.9	6.7	...	13.1	2.5		30.8	10.9	...

Descriptive	Description of monthly data (5 yrs)			Oct.	Nov.	Dec	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 220–730 ft	Monthly means	Rainfall	Amount	2.7	2.2	3.8	3.3	3.1	3.6	3.1	4.0	4.0	4.1	6.3	2.9
		Run-off	Amount	0.6	0.6	1.4	1.7	2.3	2.3	2.1	1.4	0.9	0.7	1.4	0.8
Mean temp, 1898–1910 at Petersburg, Va (30 mi southeast of gaging station). Jan., 39° F July, 78° F.	Monthly max	Rainfall	Year	1902	1902	1901	1903	1903	1903	1901	1901	1902	1905	1901	1901
			Amount	7.6	4.0	8.1	4.8	5.2	6.0	6.0	7.3	5.0	7.0	13.1	4.2
		Run-off	Year	1902	1902	1902	1903	1903	1903	1903	1901	1903	1903	1901	1904
			Amount	1.4	1.3	2.5	2.7	3.6	4.0	3.7	3.2	1.5	1.3	4.0	1.2
Run-off for driest 3 yrs, 1902–05, 48.0 in. Aver, 16.0 in.	Monthly min	Rainfall	Year	1901	1901	1903, 1904	1904	1901	1905	1904	1902	1901	1902	1904	1903
			Amount	0.4	1.1	1.9	2.0	0.9	2.4	1.1	1.7	3.2	1.9	2.7	2.3
		Run-off	Year	1900, 1905	1900	1900	1904	1901	1901	1904	1902, 1903, 1904	1905	1902, 1904	1904	1902
			Amount	0.3	0.3	0.6	0.6	0.4	0.8	0.8	0.9	0.5	0.4	0.5	0.4

METHODS OF MEASURING STREAM FLOW

***Gage Hight.** Two necessary factors for determination of flow during a given period are: (*a*) Daily gage hight referred to an arbitrary datum; (*b*) discharge for various gage hights. This assumes the discharge a function of the gage hight. From (*a*) and (*b*), a rating table is constructed. To compute the flow for any given place and time the area of the water cross-section must be determined together with the corresponding average velocity.

Gaging Stations. A thorough knowledge of the physical characteristics of a stream is necessary to properly select gaging stations. A good station for higher flows is often useless for the smaller flows, while a station well adapted to medium flows has proved worthless for the extreme stages. Rarely can a station be used for all stream stages with equally good results.

Soundings.† The ideal measuring section should be perpendicular to thread of current, and where conditions of bank and bed, above and below, are permanent. This section is divided into partial areas by perpendiculars terminating in surface at points where observations of depth and mean velocity in the vertical are made. The measuring points should be spaced to show any irregularities in cross-section or velocity. For repeated measurements, points should be permanently marked. In sounding with a rod, take care it does not sink into bed of stream, and that reading is not too high on account of water running up on it. Soundings with a line are most readily taken as follows Weight and line are lowered until weight rests on bed directly under measuring point; with line taut, a point is marked on it opposite a fixed point on the bridge or cableway car from which measurements are being made; weight is then raised until it just touches surface of water and length of sounding line that passes fixed point is measured.

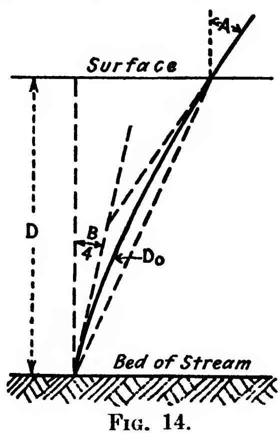


FIG. 14.

Soundings should be made by a weight attached to a line, the weight having a pointed end upstream to offer least resistance to the current. The following formula‡ gives the corrected depth D (D_0 being measured depth) when the angle of deviation of the sounding line from the vertical has been observed as A, Fig. 14:

$$D = D_0(1 - 0.000015\,A^{2.05})$$

This formula is derived from observations by F. C. Shenehon on the Niagara river. It gives too small a negative correction. The following method gives better results. Suspend the weight so that it is just immersed, and observe the angle, B, which the line makes with the vertical. Assume the sounding line makes an angle $\left(\frac{B}{4}\right)$, with the vertical when the sounding is being made. (See Fig. 14.) Observe the angle A as before.

$$D = D_0 \cos\left(\frac{A + \frac{B}{4}}{2}\right), \text{ or } D = D_0 \cos\left(\frac{A}{2} + \frac{B}{8}\right)$$

Gaging by Floats. Velocities of streams may be obtained for rough approximations of discharge by placing in midstream floating bodies of about the specific gravity of water, and noting the time taken to traverse a measured distance. It must be borne in mind that the velocity is greatest at the surface of the water in midstream. Experiments show that the mean velocity for a cross-section is about 83 per cent. of this maximum. A fair allowance must be made for local conditions altering this percentage. The cross-section may be estimated by measuring the depth at equal distances across, plotting a profile, and planimetering the plotted area; or by taking the average of these depths, and multiplying by the width.

Velocity Variation in Cross-section.* The ratios of the average velocities, V_a, in the vertical cross-section to those at the surface, V_s, may be expressed:

$$\frac{V_a}{V_s} = 0.79 + \frac{2.80}{\frac{W}{D} + 8}.$$

This formula was tested by comparison with values determined by 43 measurements, at many places on streams widely different in physical characteristics; the ratio of width to depth $\left(\frac{W}{D}\right)$ varied from 7 to 110; the flow, or discharge, from 850 to 62,000 cu. ft. per sec. Errors by the formula were as follows, some being plus, some minus: In 5 cases, no error; 11 cases, 1 per cent.; 7 cases, 2 per cent.; 8 cases, 3 per cent.; 5 cases, 7 per cent., the maximum.

Velocity* may be computed with the aid of a current meter by 6 methods: (1) Vertical velocity curve; (2) the single point method (one gaging in each vertical about 0.6 depth of the stream below the surface); (3) top and bottom method (1 ft. below top and 1 ft. above bottom) the average being used; (4) vertical integration method; (5) two-tenths-eight-tenths method; (6) sub-surface method.

(1)† Measurements are usually made just beneath surface, at 0.5 ft. below surface, at each fifth to each tenth of depth, and as near the bottom as possible. These measured velocities, plotted with depths as ordinates and velocities as abscissas, define vertical velocity-curve, which shows velocity at every point in vertical, and from which mean velocity can be determined by dividing area bounded by curve and axis of ordinates by mean depth. The following three methods are used in this determination:—(*a*) Determine area with planimeter; (*b*) divide area into sections of equal depth, usually ten; take mean of velocities at mid-points of sections as mean velocity. (*c*) Divide area into sections of convenient depth, which will be equal except bottom section, which may have odd depth. If bottom section is same depth as others, take mean of middle ordinates. If bottom section is odd depth, multiply its mean velocity by ratio of its depth to sum of middle ordinates of other sections, and divide by number of sections. Takes too long for ordinary use; it should be used only as a standard. In (2) for flood measurements, the meter is held about 1 ft. below the surface and a coefficient of 0.85 to 0.90 is applied to reduce the observed velocity to the

*J. C. Hoyt, E. N., Jan. 14, 1904.
†J. C. Hoyt, Trans. Am. Soc. C. E., Vol. 66, 1910.

mean for that vertical. In (4), the mean velocity for the vertical is determined by lowering the meter at a slow uniform speed from the surface to the bed, and then raising it to the surface. The velocity as indicated by the mean number of revolutions per second is taken as the mean velocity for that vertical. Special care has to be taken in lowering to avoid increasing the number of revolutions, by the vertical motion. This method is used to good advantage in high-water measurements, where the high velocity makes it impossible to hold the meter at any given point. It is also used under ice or where conditions are such that the point method is unreliable.

The "six-tenths" method (see (2)) has been superseded on the Catskill Waterworks of New York City by the "two-tenths and eight-tenths" method, which is theoretically and practically correct.

*(5) In two-tenths-eight-tenths method, observations are taken at depths from surface of 0.2 and 0.8; mean velocity is taken as mean of velocities at these two points. Method is based on theory that vertical velocity-curve is a parabola. Experience proves this method gives more consistent results than any other except vertical-velocity-curve method.

*(6) In the sub-surface method, measurement is made at from 0.5 to 1 ft. below surface, depending on depth of stream; meter is held at sufficient depth to be out of surface disturbance. When this method is used, velocity must be reduced by coefficient to obtain mean velocity; coefficient varies between 78 and 98 per cent., depending on depth and velocity of stream. The deeper the stream and the greater the velocity, the greater the coefficient. For average streams in moderate freshets use 90 per cent.; in flood, 90 to 95 per cent.; at ordinary stages, 85 to 90 per cent.

*Independent discharge measurements, as a rule, are of little value unless taken at stages known to be either extremely low or extremely high. In ordinary work it is necessary to make a series of measurements which, with daily gage hights, make possible computation of total flow and its distribution.

***A current meter** comprises: (*a*) wheel arranged so that when suspended in flowing water pressure of water causes it to revolve; (*b*) device for indicating number of revolutions. Relation between velocity of water and revolutions of wheel is determined by rating each meter, *i.e.*, moving the meter through still water over a known distance in observed time, and noting the corresponding number of revolutions. The counting of revolutions may be done (1) by autographic apparatus; (2) by counting the number of buzzes caused by making and breaking the circuit; (3) by some sort of telephonic apparatus. Current meters are direct and differential acting, depending on whether water, in revolving wheel, does or does not exert force which tends to retard motion of wheel. Wheels of direct-action meters consist of flat or warped-surface vanes, set on horizontal axis, which revolve by direct pressure of water against blades. In this type friction increases as velocity decreases. Principal direct-action meters are the Haskell and the Fteley. Wheels of differential meters are made up of cups which revolve on vertical axis. Pressure of water on cups being less on convex than concave side, causes wheel to revolve. In this type, friction increases with velocity, but increased frictional effect is

*J. C. Hoyt, Trans. Am. Soc. C. E., Vol. 66, 1910.

overcome by increased motive power due to velocity. The area of the cross-section offers larger resistance to current than in direct-action meter, but wheel revolves more slowly owing to retarding motion caused by resistance on convex side of cups. Principal differential meters are the Price and the Ellis. After experimenting with various types the Geol. Survey developed a meter combining essential features of Price acoustic and large Price electric meter, known as small Price meter (see Fig. 15). It is used on the Catskill Waterworks, by U. S. Geol. Survey and others.

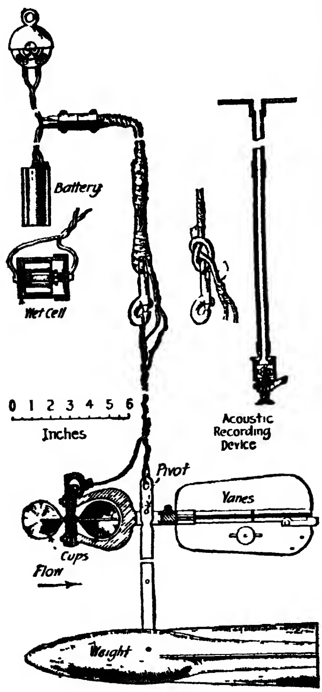


FIG. 15.—Small Price current meter. (Hoyt, T A S C E, Vol 66, 1910)

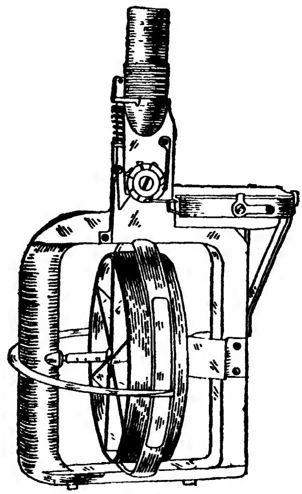


FIG. 16.—Fteley-Stearns current meter.

Fteley-Stearns type of current meter, embodying improvements suggested by A. T. Safford, Consulting Hydraulic Engineer, Lowell, Mass., is shown in Fig. 16. It has non-corrosive iridium bearings, nearly frictionless; rear bearing is specially protected against sawdust, etc. The wheel is heavier than in previous instruments of this type, is 5 in. in diameter instead of 3½ in., and has six vanes instead of eight; the pitch of the wheel is 2.3 ft. The axle and wheel are built to withstand velocities as high as 13 ft. per sec. Special attention is given to rigidity by carrying a horizontal beveled bar around in front of the wheel, securely fastened at both ends to the circular bar inclosing the wheel. All framework is made of thin knife-edge blades. The automatic counter is driven by inclosed bevel gears. The counter-box is reinforced and protected by the rod brace as shown. It is claimed that the meter begins to record at a flow of 0.1 ft. per sec., and is accurate from 0.2 ft. per sec. The Fteley-Stearns type of meter is made by the Buff & Buff Mfg. Co., Boston, Mass.; this model, No. 8, is listed at $151. E. N., Feb. 18, 1915.

Current Meter Accessories. A dry battery cell in the pocket and a single-wire telephone is better than the old-time buzzer. The current meter is sus-

pended from a single fine piano wire, from which the meter is insulated. A short length of fine insulated copper wire is led from the lower extremity of the piano wire to the forward terminal of the meter. One terminal of the dry battery is attached to the telephone receiver and thence to the piano wire; the other terminal of the battery is grounded. At each revolution of the meter wheel, current flows from the battery to the telephone, to the meter, thence, by grounded connection, back to the terminal. For the portion not submerged, the usual insulated wire may be substituted for the piano wire as it is easier to handle. With the broad surface offered the current of the stream by the ordinary heavily insulated wire cable, it is next to impossible, even by attaching a 30-lb. weight and stay lines, to keep the meter below surface at flood times. It is difficult also to determine the depth and location of the meter. Piano wire with torpedo-shaped weights of 7.5 and 15 lbs. in place of the old flatiron weights, obviates these difficulties and the meter will sink in an almost vertical line. A commutator box can be attached to make counting easier for high velocities, as it reduces the actual velocity of the meter wheel 5 times.

Note Keeping.* In counting the revolutions of the meter the results may be conveniently set down in a notebook in the following form, especially in rapid or long-continued counting:

||| ||| ||| | ||| ||| ||| | ||| ||| ||| | ||| ||| | = 37

Short strokes in groups of three and the tenth long, make easy checking, and rapid summing up possible.

Diaphragm Method, invented by Prof. Erik Andersson, Univ. of Stockholm, has been used in Europe since 1905. It is a modification of the float method, in which velocity is integrated over the entire channel area. Its use is restricted to measurement of water flowing in a channel of uniform cross-section. The apparatus consists of a vertical diaphragm, fitting the canal section with clearances not exceeding $\frac{1}{2}$ in., and suspended from a car which runs on rails on the canal walls. The apparatus is made light and of low friction, by use of steel tubing as a diaphragm frame, over which is stretched oiled canvas, and by use of ball bearings for the wheels. The diaphragm swings about a horizontal axis in the direction of the current so that it can be immersed or withdrawn without undue disturbance of the water surface. A clutch holds the diaphragm rigidly vertical while measurements are being made. The operation consists in running a short distance after lowering the diaphragm, to make sure the car attains the velocity of the current. Time of passage of the car over a measured space is then recorded. This observation, with a slight correction for water escaping through the $\frac{1}{2}$-in. clearance, gives the stream velocity. Chief advantage of this method is rapidity with which measurements can be made, especially if the slight correction for clearance be neglected; no laborious computations are required. Disadvantages are that a channel of uniform section must be available; cost of installing apparatus is high, and method is limited to moderately large quantities. Where a suitable channel is not available, a wooden flume may be built. Gaging length should be 50 to 100 ft., but

* C. E. Grunsky, "Methods of Measuring Stream Flow in U. S." (German).

a length of 10 to 13 ft. has been used. As much as 830 c.f.s. have been measured. From European practice, C. R. Weidner concludes: (1) Diaphragm gagings agree with meter gagings within 1 per cent. (2) Rating canal should be long enough to overcome effect of pulsations in the stream. (3) Diaphragm should be in a vertical position during the test. (4) Results with perturbed flow are as good as with parallel flow. For epitome of results, 1905–1914, and bibliography, see Bull. 672, Univ. of Wisconsin.*

Gaging Brooks of Small Flow. On brooks of small flow, a channel can be formed for current meter readings. Tests of accuracy showed discharge of 0.13 sec. ft. by meter and 0.135 by positive measurement (a box). Another reading gave 0.15, positive, and 0.14 metered. Also a small weir can be set.

High Velocities in Stream Channels. The highest velocities which the Geological Survey has measured in natural channels were taken on Grand River at Glenwood Springs, Colo.; several measurements ranged from 10 to 15 ft. per sec. These measurements were not made at the maximum stage and it is probable that the velocity of this stream exceeds 20 ft. per sec. at times. As Grand River is a typical mountain stream, there are probably many other streams which have velocities as high as 20 ft. (Mean velocity of cross-section.)

* "Diaphragm Method for the Measurement of Water in Open Channels of Uniform Section," by C. R. Weidner, 1914.

CHAPTER IV

GROUND WATER

OCCURRENCE

Absorption of Rainfall by Ground. Ground receives the greater part of the rainfall, probably nearly 80 per cent. in eastern U. S. and 90 or 95 per cent. in much of the West. Water that enters sands and gravels generally moves toward streams, but in regions where the rainfall is small, gravels may absorb water from streams which rise in regions of greater rainfall.

Most favorable ground-water areas are the deep water-bearing strata beneath broad plains that were created during the glacial epoch by out-washes of coarse sands and gravels from the terminal moraines of glaciers.

Depth of Water-table. The upper limit of water-bearing strata is known as the water-table. Its depth varies with: (a) porosity of the catchment area; (b) amount of rainfall; (c) topography. In regions of low rainfall and low relief, the water-table is deep-seated and relatively horizontal. In regions of greater rainfall and greater relief, it is relatively near the surface and tends to follow the topography. If a valley cuts a water-table, the ground water moves toward the valley, forming springs, as shown in Fig. 17.

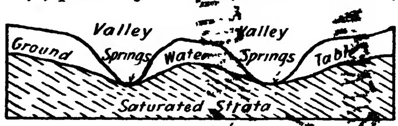


FIG. 17.

Relation of Water-table to Vegetation. Investigations have shown that when the ground-water surface is 5 ft. or more below the surface of coarse soils, no moisture reaches the surface or the roots of vegetation through capillary action.

Fluctuations of Ground-water Level.* The annual fluctuations of ground-water level—high in summer, low in winter—are caused by the quantity of rainfall reaching the plane of saturation. Frozen ground minimizes the absorption of surface waters, although King's experiments in Wisconsin showed conclusively that some water percolates into frozen ground, possibly through shrinkage cracks.

Causes† of the fluctuation of ground water-table on Long Island are: (1) Natural: Rainfall, sympathetic tides, thermometric changes, barometric changes. (2) Artificial: Dams, pumping. Tests on Long Island indicate that the deeper below the surface and the higher above sea level the water table is, the less rapidly it responds to rainfall influence. The retardation was entirely out of proportion to the thickness of the unsaturated beds above the water table.

* Burr-Hering-Freeman report on Water Supply for New York, 1903, p. 825.
† U. S. Geol. Survey, Prof. Paper # 44, 1904, p. 69.

Underground Water in Rocks.* Experience does not prove the assumption that all rocks are saturated below a moderate depth. In the Pennsylvania and New York oil regions, rocks practically destitute of water are encountered at a few hundred feet. These include coarse-grained sandstones capable of holding large quantities, but which are dry. Very rarely is fresh water found below the dry rocks. Wells have been drilled several thousand feet deep without encountering water below the first few hundred feet. These facts show the fallacy of the idea that there is plenty of water if one only goes deep enough. That great underground lakes exist is extremely improbable.

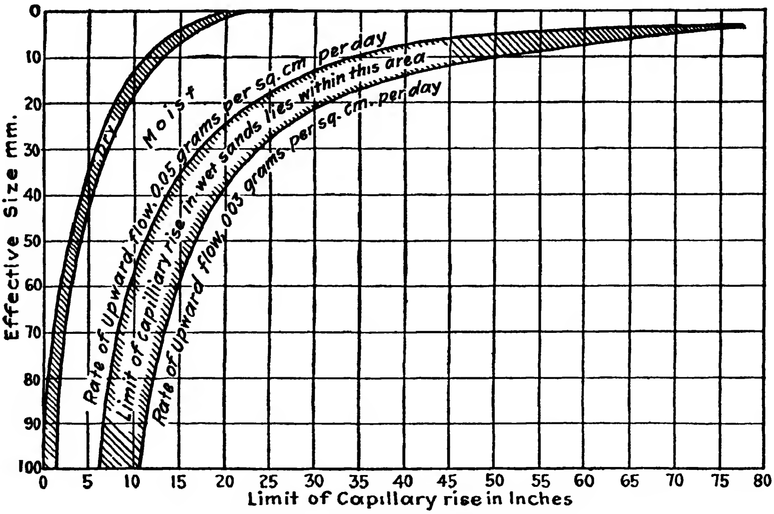


FIG. 18.—Relation of capillary rise to effective size.†

Purity of Ground Water.† In a sandy soil, like that on Long Island, probably all ground water found under natural conditions at a depth of more than 10 ft. is safe for drinking, provided it does not receive immediate subsurface pollution. Tubular wells will yield safe water, but open or dug wells are too liable to pollution from above to be used in crowded districts. In several European cities, ground water is preferred to filtered river water. It should not be drawn too near the surface on account of sewage nor too near the sea on account of salt. See also p. 238.

Yield. The normal yield of the ground water catchment area of the Ridgewood (Brooklyn, N. Y.) system, is about 900,000 gals. per day per sq. mi., or 43 per cent. of the average rainfall. European watersheds of similar character, on which the rainfall is much less than on Long Island, have averaged 40 to 50 per cent. of the rainfall for years.

Velocity Measured.‡ Slichter observed an average velocity of 7.4 ft. per 24 hrs., and a maximum velocity of 22.9, of underflow in Arkansas river sand. G. E. P. Smith found a velocity of 400 ft. per day in the underflow of an Arizona stream. Character of deposit has large influence.

* See also pp 76 and 77.
† Burr-Hering-Freeman report on Water Supply for New York, 1903
‡ Trans. Am. Soc. C. E., Vol. 73, 1911, p. 195. See also C S. Slichter in Water Supply Papers of U. S. Geol. Survey.

Artesian Conditions. In order that a well may flow, the following conditions must be satisfied: (1) Sufficient rainfall. (2) Relatively porous beds suitably exposed to collect and transmit the water. (3) Less porous or relatively impervious layers so placed that they may confine the water collected. (4) The level of the ground water at the source should be at a sufficient hight above the mouth of the well to compensate for loss of head due to resistance and leakage

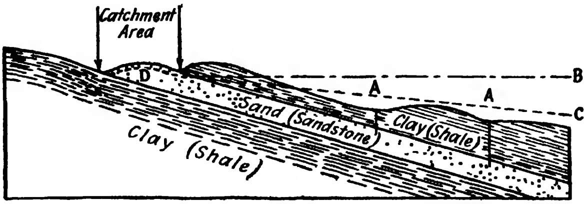


FIG. 19.—Ordinary conditions producing artesian wells. A, wells; B, head of water if there is no loss by resistance and leakage; C, actual head or hydraulic gradient; D, ground-water table at outcrop.

WATER-BEARING FORMATIONS

(U. S. Geol. Survey, Water Supply Paper No. 255, Myron L. Fuller, 1910)

Geologic Considerations. Observing well drillers will take into account surface drainage of the region, presence or absence of forest which may influence run-off, existence of springs as evidence of surplus water, and depth and character of soil overlying the rock, especially structure of the soil cover. If water-laid, and therefore stratified and regular, accurate and quick deduction can be made as to depth and availability of ground water. If material be wind-laid, and therefore less regular, deductions are uncertain; likewise in regions formerly covered by ice (glacier), where it may be still more irregular. All ground water has more or less intimate relation to surface drainage and topography.

Beds of sand and gravel are very porous, as much as 30 per cent. of the volume of some being free space, so that saturated layers penetrated by wells yield copious supplies. This water is in most places of good quality, but in some wells is greatly mineralized from the more soluble rock fragments that constitute the deposits. In passing downward through finer sands, surface water may be filtered naturally. In coarse sands and in gravels water passes downward more rapidly and conditions are less favorable for filtration. In general, water from sands and gravels far below the surface is pure. Because sands and gravels yield readily, wells close together may affect one another, those which derive supply from the lowest points drawing from the shallower wells. To procure permanent supplies a well should penetrate below the level to which the water surface sinks in driest seasons. Wells in sand and gravel should have large enough diam. to avoid clogging around casing, which seems inevitable in wells of small diam.

Pure clay is nearly impervious to water and contains little or none that can be utilized; water is frequently reported in clay, but as a rule it comes from

more or less sandy layers within a clay bed. In some places sand that approaches clay in fineness and is mistaken for clay yields much water. Clay is of great importance as a confining layer to porous sands. When necessary to obtain water from clay, the well should be as large as practicable and deep enough to provide ample storage, for clay yields a small quantity, and very slowly. Dug wells are usually most satisfactory where clay is near the surface, but should be carefully covered and protected from pollution.

Till is a heterogeneous mixture of clay, sand, gravel and bowlders. In texture it ranges from very pervious to impervious, according to content of sand or of clay. In few places is it definitely bedded. Water generally occurs in it in minute, more or less tubular, channels, but occasionally is distributed through interstratified sandy beds. In finer, more loamy phases the supply is not abundant, but coarser portions furnish water more plentifully. In the aggregate, till yields a large quantity, and where sufficiently thick forms a most convenient and accessible source, but because so irregularly disposed the success of wells varies greatly. Water is usually close to the surface, but better can be obtained by casing off upper water-bearing beds and extending the wells to greater depth. In general, wells of large diam. are most satisfactory. Till is widely distributed over northeastern and north-central U. S. in areas covered by glacial drift. Southern border of glacial deposits extends from Martha's Vineyard, Mass., through Long Island and northern New Jersey, across northern Pennsylvania into southwestern New York; thence southwestward across Pennsylvania and Ohio, southern Indiana, Illinois, and central Missouri, and northwestward into Canada.

Sandstone is, of solid rocks, the best water bearer. Water from sandstone is of better average quality than that from any other material except sand and gravel. In the Appalachian Plateau—the bluegrass district of Kentucky, Nashville basin, and Cumberland and other plateaus of Tennessee, West Virginia, western Pennsylvania, and New York—sandstone is the chief water-bearing sedimentary rock. Porous sandstones also underlie the great plains of the Dakotas, Kansas, and Nebraska, and yield artesian flows throughout extensive areas. Over much of the region farther south they also furnish abundant underground water. In some places *conglomerate* yields considerable water, although absorptive capacity is not so great as of sandstone; it is not so widely distributed. *Quartzite* is a metamorphosed sandstone, the spaces between grains being filled by hard, siliceous matter. Because of this filling of the pores there is little chance for water, hence it is not commonly an important source. Old sandstones, shales, and other sedimentary rocks altered to quartzites or slates lie near the surface in the Appalachian Mountains, Ozark and Superior highlands, and Rocky Mountains, and on the borders of these higher lands yield some water, but generally both occurrence and quality are uncertain.

Shale is a poor water bearer, but may yield from bedding, joint, and cleavage planes, and other crevices; most important as a confining layer to prevent escape of water from interbedded porous sandstone.

In **limestone** water occurs mainly in channels and caverns formed by solvent action along joint or bedding planes. These are very irregularly distributed

and location can seldom be determined by examining the surface, but most deep wells encounter one or more such passages at relatively slight depth. Adjacent wells, only a few feet apart, may obtain very different results. Water is hard, and likely to be polluted, since much of the water in limestone has found its way downward through sink holes and carries surface wash. Water-bearing limestones occur in southern and southeastern U. S. and in the Appalachian Mountains, but owing to poor quality, recourse is usually had to springs or to wells sunk in other rocks.

Granite and gneiss are dense and have very small pore spaces. *Schist* may contain water that has penetrated along foliation planes. Most wells in crystalline rocks obtain water, if at all, within 200 or 300 ft. of the surface, and it is generally useless to go deeper than 500 ft., although in some places, as at Atlanta, Ga., water supplies have been obtained at depths as great as 1600 ft. Joints in crystalline rocks usually form complex systems of intersecting planes, and polluted water may pass from the surface along joints until finally it reaches the well at a depth of many hundred feet.

Basin and Stream Deposits. Along northern portion of Atlantic coastal plain are extensive lake and stream deposits of clay, sand, and gravel, in which water is usually plentiful and good. In lower Mississippi Valley fine silts of the flood plain are saturated a short distance below the surface and furnish abundant supplies to small wells; chief drawback is the unusually small mesh of screens necessary to exclude fine sand. In most places a lens of coarser sand or gravel can be located, and drawn on. In many parts of the arid region of western U. S. great basins or valleys are deeply filled with sediments brought down by streams. In some places these contain water at shallow depths, and form the chief source of supply throughout much of the Great Basin region of Utah, Nevada, southeastern Oregon, and southeastern California. In the more favorably situated of these desert valleys, notably Coachella Valley, in southeastern California, the deeper water of the unconsolidated deposits may be under sufficient artesian head to yield flowing wells. In the great central valley of California and on the coastal slopes of the Pacific States are deep alluvial deposits of importance as sources of water. Those of the coastal plain of southern California have been extensively tapped to obtain water for irrigation, and water is drawn from similar deposits around Puget Sound.

FLOW OF UNDERGROUND WATERS

Laboratory Experiments Inadequate. Laboratory experiments on flow through unassorted sands do not consider the modifying influences of frost, vegetation, or the intermittent and variable intensity of application of water. Permeability of soil has been found to be one-fourth as much as fine sand; when the organic matter was burned out, it was about one-half as permeable. A disturbance of the medium has a marked effect on the rate of flow. Even a very slight scratching of the surface seriously affects the discharge. With pure blow sand, percolation decreased from 15 to 5 cu. ft. per sec. due to a slight disturbance of the surface. The manner of stratification also has a modifying influence.

Formulas. The principal formulas now in use are:

Hazen's: $V_m = C_p d^2 S \frac{t_f + 10}{60}$. (Report, 1892, Mass. State Bd. of Health.)

Slichter's: $Q = 0.2012 S \frac{d^2 A}{uK}$. (U. S. Geol. Survey, Water Supply Paper No. 67, 1902.)

Darcy's: $Q = C_p SA$

Forchheimer's: $S = aV_m + bV_m^2$

Lueger's: $V_d = 283\ dS$. (König, Wasserleitungen, 1907, p. 164.)

a = a constant, the conditions governing it not being explained.
b = a constant, the conditions governing it not being explained.
C_p = constant, depending on porosity of the medium.
d = effective diameter of sand grain, millimeters; varies from 0.1 for very fine sand to 3.0, fine gravel. See also p. 739.
A = area of total cross-section yielding water, in square feet.
A_v = area of the void section, square feet.
S = slope head or difference of elevations of the water at the two points considered ÷ distance traveled, measured along path of travel, $= \frac{h}{L}$.
K = constant, dependent on porosity of the medium
Q = cubic feet per min. of water yielded by the entire cross-section.
t_f = temperature of the water, in deg. Fahr.
T_c = temperatue of the water in deg. Centigrade.
u = a constant known as the coefficient of viscosity, which takes account of the friction between particles of liquid.
V_m = velocity in meters per 24 hrs., at which water issues from the entire cross-section.
V_d = V_m expressed in feet.
V_v = velocity, in feet per 24 hrs., at which water travels through the voids.

The formula by Forchheimer of Gratz, Austria, although used in France, is of little use in United States, no working values being specified for a and b. These could be obtained by a routine test of each soil to be dealt with. Sellheim, Smreker, Masoni and Krober have also produced formulas.

Slichter's formula may be written:

$$V_d = 289.728 S d^2 \div uK;$$

and Hazen's:

$$V_d = \frac{292}{89} C_p d^2 S \frac{t_f + 10}{60}.$$

Velocity depends on the square of the effective diam., on porosity and temperature, and on slope, or pressure gradient; and is the velocity of the water issuing from the entire cross-section of the water-bearing sands. Velocity in voids is greater than that given by the formulas; but the cross-section in this case will be that of the voids. $Q = A_v V_v = A V_d$; therefore $V_v = A V_d \div A_v$. Velocities of flow which occur in seepage, percolation or in forced flow through pervious materials are affected by capillary attraction and frictional resist-

ance. Effects of these forces are coexistent, but not identical, and vary inversely as sizes and proportion of interstices between particles of alluvium. In fine clay, in which interstices are infinitesimal, it is almost impossible to force appreciable flow through a few feet thickness. If water finds passage through any part of variable density, or fissure or shrinkage crack, a barely appreciable trickle, under pressure, will rapidly disintegrate and abrade clay. In materials with relatively large interstices, friction and capillary attraction offer less resistance and water has appreciable velocity. Permissible velocity of flow must be confined to that which the material will stand without movement, displacement or abrasion.

D'Arcy, Hagen, Hazen and others found that for grades varying from very fine sand to fine gravel, velocity closely followed the law of capillary flow.

The Slope. The experiments of Hazen and Slichter indicate that velocity bears a straight-line relation to the slope. F. H. King's experiments showed that the flow of water may increase somewhat faster than the slope. Tests in India have tended to prove that the quantity of water discharged varies directly as pressure head. In 1905, Slichter published results of laboratory tests on flow through sands and gravels in both horizontal and vertical tanks, and this straight-line relation of velocity to slope was verified as thoroughly as laboratory experiments can be applied to natural conditions. This relation is verified by still later experiments. It has been established that percolation of water through sand and gravel is in direct ratio to the heads lost by friction. In discussing the Bohio dam, Geo. S. Morison applies Hazen's formula to seepage at a slope of $90/2500 = 0.036$. To obtain the slope, S, ascertain the difference of the average elevations of ground water at the two points under consideration (the average allowing for daily fluctuation of level) and divide this difference by the shortest distance between the points considered. If the actual distance traversed by the water is known, substitute it for "shortest distance."

Viscosity. The coefficient of viscosity is defined as the force necessary to maintain unit difference of velocity between two layers of water unit distance apart. It depends on the temperature. Natural springs flow more freely at night than in the day. Unwin's experiments have shown that surface friction of solids on liquids is diminished 1 per cent. for every 5° rise of temperature. Hazen found for sands a 3° rise of temperature increased the flow 5 per cent. Where water passes a long distance through a medium it will have the same temperature as the medium. Ground-water temperature is generally 50° F. The smaller the opening, the more important is viscosity. At 200°, five times as much water will pass through a capillary tube as at 32°.

Formulas for effect of temperature on flow: (See Fig. 20.)

Slichter:	$u = 0.0178 \div [1 + 0.0187(t_f - 32)]$.
Meyer:	$u = 0.0183 \div (1 + 0.0369T_c)$.
Slotte:	$u = [0.5212 \div (26 + T_c)] - 0.00131$.
Poiseuille:	$u = 0.017831 \div (1 - 0.0336793T_c - 0.0002203936T_c^2)$.

Porosity. Of more importance than viscosity is porosity, or the relation of the void spaces in the medium to the total volume. Porosity depends on variation of sizes and manner of packing. Experiments on unsorted gravels, varying from 2½ in. to ½ in., heavily rammed, prove that 28 per cent. voids is about the lowest obtainable. The maximum porosity possible, in a mass of equal spheres, is 48 per cent. Porosity of quartz sand varies from 30 to 40 per cent., and that of clay loams between 40 and 50 per cent; water-bearing sands and gravels, 10 to 30 per cent. The *K* of Slichter's formula and C_p of

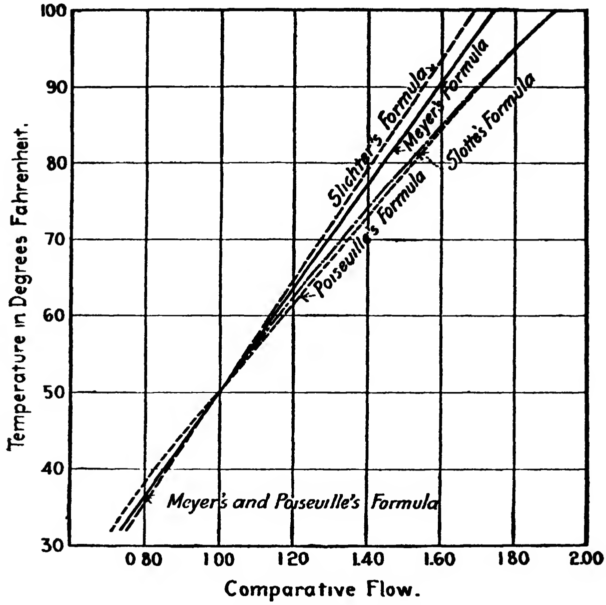


FIG. 20.—Comparison of viscosity formulas.

Hazen's, take account of porosity.* Hazen's formula (Mass. State Board of Health, 1892, p. 539) applies to sands with effective diam. from 0.1 to 3 mm.; for gravels of larger size, friction varies in ways not expressible by a general formula.* As the size increases beyond this point, the velocity with given head does not increase as rapidly as the square of effective size; and with coarse gravels, the velocity varies as the square root of the head instead of directly with the head, as in sands. The influence of temperature also becomes less marked with coarse gravels. Whenever properly applied, the Hazen formula has proved reliable; C_p rarely falls below 400, even for old and dirty sands and rarely rises above 1200, lying between 700 and 1000 in most cases. This formula is not to be applied to clay, hardpan, soil, etc. Voids may be measured by ascertaining the volume of water contained in a sample. This method is subject to errors due to air entrained in the water, and the absorbent action of the stone. Better, weigh a known volume and from the known weight of an equal volume of solid stone, compute voids.

* Trans. Am. Soc. C. E., Vol 73, 1911, p. 200.

Independence of effective diam. and porosity is illustrated in the following table from Water Supply Paper #67.

Table 27. Relative Flow of Water through Sands of the Same Effective Size, but Packed to Possess Different Porosities

Porosity, or per cent. voids	Relative flow compared to a porosity = 32 per cent.
30	0 8
32	1 0
34	1 2
36	1 5
38	1 8
40	2 1

Pore spaces of soils, per cent. of volume; Illinois prairie soil, 55; East Windsor, Conn., clay soil, 48; coarse river sand, 38 to 41; subsoils, 35 to 43; blowing sands 45. (Rafter & Baker, "Sewage Disposal in U. S.," 1894.)

Table 28. Porosities* of Some Media Encountered by Ground Waters

Rock or earth	Quarts of water per cu ft	Porosity, per cent		
		Minimum	Maximum	Average
Granite, schist and gneiss	0 003–0 06	0 02–0 4	0 6–1 9	0 2–1 2
Gabbro	0 06†	-----	-----	0 8
Diabase .	0 07†	0 9	1 1	1 0
Obsidian. . .	0 04†	-----	-----	0 5
Sandstone	0 5–1 5	3 5–4 8	22 8–28.3	10 2–15 9
Quartzite	0 01–0 06†	-----	-----	0 2–0 8
Slate and shale ...	0 30†	0 5	7.6	4 0
Limestone, marble, dolomite	0.06–2.5	0.5	13.4	4 8
Chalk	2 0	-----	-----	53 0
Oolite	0 54†	3.3	12 4	7 2
Gypsum	0 19†	1 3	4 0	2 6
Sand (uniform)	2.5	26 0	47 0	35 0
Sand (mixture)	2 85†	35 0	40 0	38 0
Clay..	3 35†	44 0	47 0	45 0
Soils	4 12†	45 0	65 0	55 0

Uniformity Coefficient. The Massachusetts tests indicated that the finer 10 per cent. of the sand grains control the transmitting power. The uniformity coefficient is found by dividing the size of grain separating the coarser 40 per cent. from the finer 60 per cent. of the particles, by the size of grain separating the finer 10 per cent. from the coarser 90 per cent. A rough estimate of the open space can be made from the uniformity coefficient. Sharp-grained particles having a uniformity coefficient below 2 have nearly 45 per cent. voids as ordinarily packed, and sands having a coefficient below 3, as they occur in banks, or artificially settled in water, will usually have 40 per cent. open space. With more mixed materials, the closeness of packing increases until, with a uniformity coefficient of 6 to 8, only 30 per cent. open space is left. With round-grained water-worn sands, the open space has been observed to be from 2 to 5 per cent. less than for corresponding sharp-grained

* M. L Fuller, Water Supply Paper #160. † Computed from average porosity.

Table 29. Percentage of Absorption (by Volume) of Various Geological Strata (Mead, Notes on Hydrology)

Formation	Location	Per cent.
Sandstone	Grand Beauchamp, France	13.2
Sandstone, another specimen	Grand Beauchamp, France	4.4
Calcareous freestone	Grand Beauchamp, France	18.0
Lower tertiary sandstone (pure quartzose)	Grand Beauchamp, France	29.0
Upper chalk	Ivry, France	24.1
Devonian limestone	Boulogne, France	0.1
Oolite sandstone	Cheltenham, England	24.0
Oolite limestone	Cheltenham, England	12.2
Old red sandstone	Gloucestershire, England	11.6
Hornblende granite	East St. Cloud, Minn	0.4
Gabbro .	Duluth, Minn	0.3
Dolomite	Joliet, Ill.	1.1
Limestone	Quincy, Ill	0.6
Limestone	Quincy, Ill....	1.4
Sandstone.	Fond du Lac, Wis.	4.8
Dolomite	Lemont, Ill	1.1
Dolomite..	Winona, Minn. .	4.8
Dolomite	Red Wing, Minn..	2.5
Dolomite	Mantorville, Minn.	5.5
Limestone	Big Sturgeons Bay, Wis.	0.25
Sandstone	Fort Snelling, Minn	6.3
Sandstone	Jordan, Minn..	12.5
Sand and gravel	—————————	33–40
Dry clay	—————————	12.0
Trenton limestone	Rockford, Ill	2.1
Galena limetsone	Rockford, Ill	4.2
Berea sandstone	Berea, Ohio	6.6
Bedford limestone	Bedford, Ind	4.4

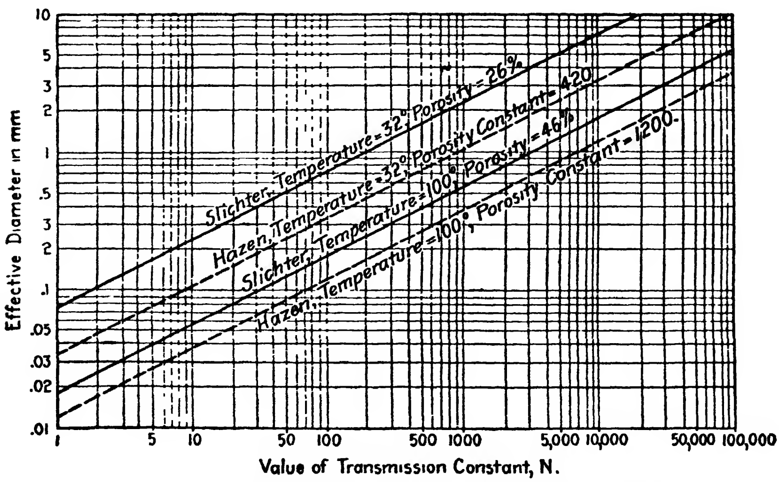


Fig. 21.—Relation of effective diameter, temperature and porosity to transmission constant.

sands.* In tests on materials for Cold Spring dam (E. N., Mar. 7, 1907) it was found that a small effective size and a large uniformity coefficient were a fairly reliable indication of small flow. The proportion of voids was found to be of little use in ascertaining the permeability of a medium. Probably one of the most influential factors in determining porosity of soils and volcanic ashes is the

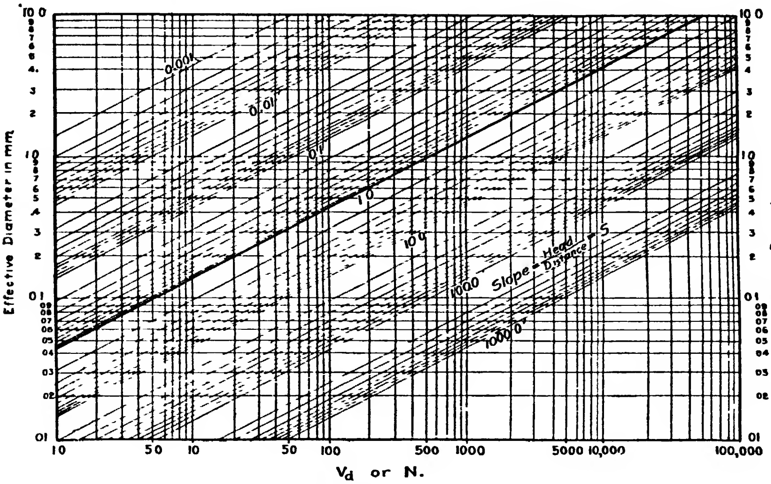


FIG. 22.—Relation of effective diameter, slope, velocity and transmission constant. N, transmission constant, is to be read from slope line 1.0 only

(Based on Temperature = 50°F, Porosity = 32 %; for other temperatures and porosities, apply corrections from Figs 23 and 24)

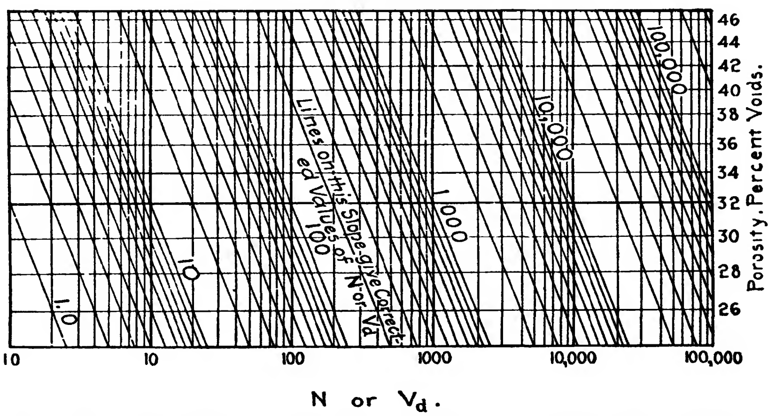


FIG. 23.—Transmission constant (or velocity). Correction for porosity.

(32 per cent considered normal)

contained vegetable matter. Although the porosity of clay varies from 40 to 70 per cent., the fineness of the grains causes slow percolation.†

Transmission Constant. All formulas except Forchheimer's are reducible to the general form obtained by Darcy: $V = NS$. N, termed by Slichter "Transmission Constant," is defined as the quantity of water transmitted in

* Report, 1892, Mass. State Board of Health. † See also p 741.

unit time through a cylinder of the medium of unit length and unit cross-section under unit difference of head at the ends. When $S = 1$, $V = N$ (Figs. 22, 23, 24).

Application of Lueger's Formula. $V_d = 283\ dS$. An underground water current 3000 ft. wide and 3 ft. deep, has slope of 0.001, passing through a medium of 25 per cent. porosity, with $d = 2.0$

$$Q = AV_d = 0.25 \times 3000 \times 3V_d = 0.25 \times 3000 \times 3 \times 283 \times 2 \times 0.001$$
$$= 1275 \text{ cu. ft. per day.*}$$

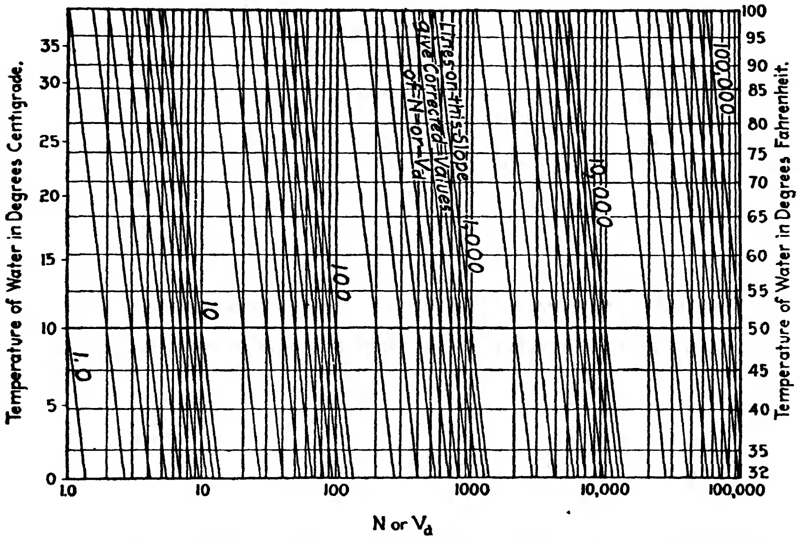


FIG. 24.—Transmission constant (or velocity). Correction for temperature. (50°F normal)

Application of Hazen's Formula. In considering flow of ground waters, refinements like influence of temperature may be eliminated (see Fig. 24). Thus Hazen's formula becomes:

$$V_d = 8200\ d^2S$$

Velocity of ground-water, through fine sand having effective size of 0.1 mm. and general slope of ground-water surface of 5 ft. per mi. (hydraulic gradient = 0.001) would be 0.082 ft. per day. Velocity through gravel having effective size of 3 mm. and general slope of water surface of 53 ft. per mile (hydraulic gradient = 0.01) would be 738 ft. per day, or approximately 0.0085 ft. per sec. Report of Mass. State Board of Health, 1892, contains results of Hazen's experiments to determine velocities of water through screened gravel, assuming 40 per cent. porosity.

The formula $V = 8200d^2S$ gives much larger results than Table 30. To modify formula to produce results given in table would involve variation of coefficient between 6222 (for d = 3 mm. and s = 0.0005) and 940 (for d = 35 mm. and s = 0.01).

* Konig, Wasserleitungen, 1907, p 165.

Table 30. Velocity of Water in Feet per Day, in Screened Gravel, Assuming 40 Per Cent. Porosity*

Slope S	Effective size, millimeters									
	3	5	8	10	15	20	25	30	35	40
0 0005	28	82	164	246	410	656	902	1,230	1,640	2,050
0 001	57	172	335	475	820	1,210	1,680	2,250	3,030	3,690
0.002	115	328	639	902	1,550	2,250	3,030	3,930	4,830	5,820
0 004	221	631	1,230	1,700	2,870	3,930	5,000	6,060	7,130	8,200
0 006	336	918	1,690	2,250	3,690	5,080	6,390	7,620	8,930	10,100
0 008	443	1,160	2,060	2,780	4,340	5,900	7,380	8,930	10,400	11,800
0 01	549	1,410	2,460	3,150	5,000	6,800	8,440	10,000	11,500	

Slichter's Method of Measuring Rate of Movement of Underground Waters. Electrical measurement of flow of underground water was developed by Slichter in 1901. Two rows of $1\frac{1}{4}$-in. drive wells are sunk across the channel of the river whose underflow is to be tested. Upstream wells are charged with strong electrolyte, which dissolves and passes downstream with moving water. Passage of electrolyte downstream is recorded by deflection of needle of recording ammeter; final arrival at the lower wells is marked by strong deflection of needle. Electrical measurement has advantage of older chlorine determination in that tedious chemical analyses are obviated, and actual movement of water is traced throughout the whole experiment. Electrolyte must possess following properties: (1) ready solubility in water; (2) chemical inactivity to dissolved matter in natural waters and to material of porous medium; (3) low coefficient of diffusion; (4) high conductivity; (5) low cost. Ammonium chlorid has given best results; with a few dry cells, a current of sufficient intensity to throw the needle of a recording ammeter is obtainable. Electric circuit to wells is made as follows: Run wire from casing of lower well to one pole of battery, with ammeter in circuit, and connect other pole both to internal electrodes of lower wells, and to casings of upper wells. Ammeter chart will show movement of ground water between wells, and abrupt indication of arrival of electrolyte at lower wells is also obtained.

Drive wells may be made of perforated pipe sections 4 to 8 ft. long to offer free passage to water. Internal electrode may be made in several insulated links with separate external wires, so that varying velocities at different depths can readily be obtained. In case of rapid flow, some caustic potash must be added to the electrolyte to spread it out into the stream wide enough to be intercepted by the lower wells. (Water Supply and Irrigation Paper #67, 1902.)

SPRINGS

Classification. Springs are classified as follows: (1) Those which bubble out of fissures of rock layers; (2) those which issue from the weathered rock strata above or beside the bedrock; (3) those which bubble up in alluvial land.

Utilizing Springs in Rock Strata. First remove all earth and weathered rock over and about the point where the water issues, so as to lay bare the bedrock. The pure water issuing from the rock is then caught in a small

* König, T. A. S. C. E., Vol. 73, 1911, p. 178.

reservoir with stone, concrete or brick walls, whence it is conducted through pipes to the point of consumption.

Earth slopes around a spring should be strengthened by sod or paving to prevent their being washed down. Make sure that the rain-water cannot find its way to the rock against which the spring house is built. Fig. 25 (from Table 1, Konig) shows a spring house without entrance, one with entrance on top, and one with a door at the side, respectively.

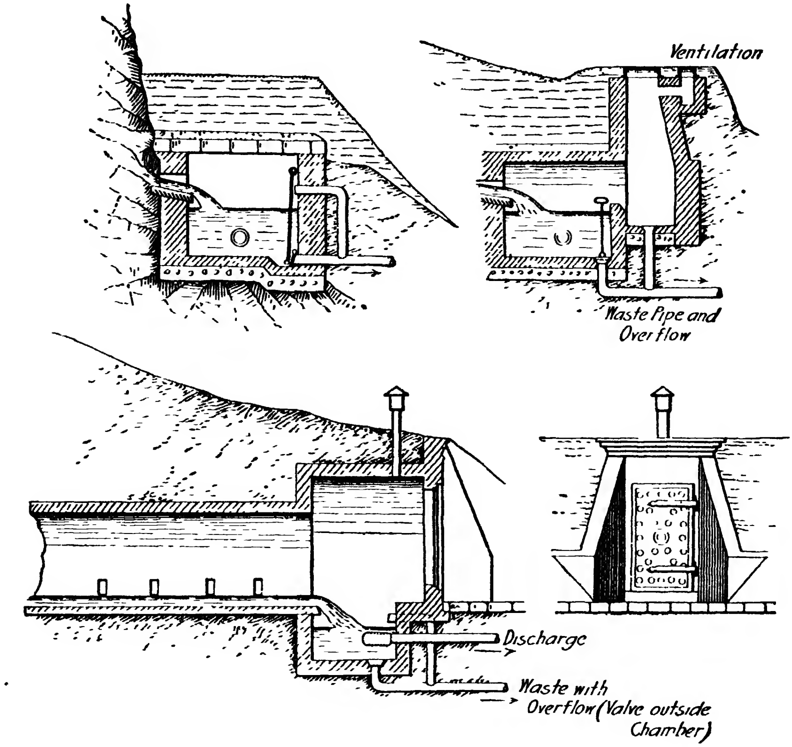


Fig. 25.—Spring houses.
(König: Wasserleitungen und Wasserwerke, 1907)

Utilizing Springs from Weathered Rock. Water issuing from partly loose and sandy rock layers often carries much sand. Springs with reservoirs which are not large or deep enough, and into which surface water passes after a rain, are often cloudy on account of their content of suspended matter. Such water should be passed through a sand-catcher, or settling basin, before entering the clear-water tank or reservoir. To keep the water pure, it is necessary to provide for this settling chamber a concrete or brick roof covered with earth, and to keep the air out. An auxiliary conduit through which the water can be passed when the settling chamber is to be cleaned, and means for emptying it, are also necessary. A suitable settling chamber is rectangular, of such a cross-section that water passes through it at a slow speed, say 10 ft. per hr., according to the nature of the suspended matter.

Utilizing Springs in Alluvial Soil. The method described in the last paragraph applies also to springs in the upper regions of alluvial land, such as mountain valleys. Springs in the lower regions of alluvial land occur mainly in trough-like depressions of large river valleys and in low-ground districts, which are either at a lower level than the ground water in the neighborhood or at place where it is dammed by deficiently pervious or impervious obstructions. On the shores of rivers and lakes, especially in inlets, ground water often issues from the soil as a spring and passes into the river or lake; springs which are covered by water reveal their existence by rising air bubbles as well as by small sand runs. Springs in low ground always rise from below, and often make themselves manifest by bubbling, so that they are comprised within the class of bubbling springs. Springs in the mountain valleys rarely behave in this way; when they issue from the soil under pressure from a higher level, they may be called fountain springs. Bubbling springs in low ground, on account of the slowness with which the water flows off, often cause a considerable accumulation of water in the form of a pool, swamp or pond, covered with plants and containing animal life. When it is possible to drain the pool artificially, the surroundings of the springs are carefully cleaned to remove mud, plants, etc., and clean solid ground is laid bare around the separate larger springs and around groups of smaller springs. The springs are then surrounded with concrete or brick walls higher than the ground and a cover is provided to prevent impurities from getting in. In most cases artificial drainage requires pumping, but with springs issuing from sand, pumps must not be operated with such force that the springs begin to yield much sand, since otherwise underground conditions may be changed in time and the outlet of the spring may be shifted. A larger area of springs, with numerous little springs, is enclosed by a wall, after the pool has been cleaned. This wall is built above the level of the surrounding ground. On the solid ground surrounding the spring area, the individual springs are connected with one another by means of little channels. If the spring area is not too extensive, it is covered with an arch roof. Very extended spring areas are enclosed only by a masonry wall, or in loose ground, by sheet piles. Channels from the individual springs communicate with a collecting reservoir, from which the water is taken when needed and which is covered and closed. Ventilation, as well as protection against frost and heat, must be taken into consideration in this case, as in all problems of spring construction, and means for emptying and overflowing should be provided.

Springs of Baden-Baden.* Along the open cleavages between the granite and colored sandstone, there appear everywhere springs or threads of water. These are either caught separately or, wherever they appear in rows, are collected in conduits. Collecting conduits with a total length of 0.75 mi., are connected together by intermediate conduits of 0.5 mi. length. The conduits are 2 ft. 4 in. wide and 5 ft. 3 in. deep. Where a stronger spring issues, a niche is broken through the wall of the conduit; opening also made in the collecting pipe below the bottom of the conduit. Smaller threads of water are collected in clay pipes along the walls of the conduits, connecting with the next niche where water flows into the main collecting conduit. The bottom of the con-

* Lueger· Wasserversorgung, 1907, p. 399.

duit serves only to collect the water dripping down from the walls; it runs from here to the spring house and then into the exit provided. The walls are built dry; the bottom, and side-walls to a hight of 8 in. are covered with cement.

Subterranean Galleries at Wiesbaden are good examples of another method of utilizing underground water supply. The water-bearing mountain is composed of fissured quartz in the interior, covered with slightly porous rock. Galleries were driven at the necessary elevation to give sufficient head for supplying water; they are pierced where flows occur, to receive the water. Total length of galleries is 3000 meters, of which 2000 are in the porous covering. Inequalities of flow are harmonized with demands for water by building doors in the galleries. These doors are of wrought iron, and made watertight by rubber gaskets on the frame. Behind these doors, erected in pairs, the surplus water is stored in ample quantities for use during the dry spells. Water is taken off by means of a valve beside the door. The local situation of the doors is determined by existing lateral outflow from the subterranean reservoirs, since the damming by means of the door should not raise the water to such a hight as to prevent free discharge from the fissures. The distance to the next doors downstream is fixed partly by the occurrence of soft formations where they can be erected, and according to the existing free flow, the hight of which must not be exceeded by the second doors. A succession of these doors in an inclined gallery gives the appearance of a flight of stairs to the water surfaces behind the doors. König, Wasserleitungen, 1907, p. 136.

Effect on Water of Covered Reservoirs. At Ridgewood (Brooklyn, N. Y., waterworks), ground water, or mixed ground and surface waters, cannot be stored in open reservoirs on account of growth of offensive microörganisms. Artificial storage reservoirs usually are unnecessary for a properly-designed ground water-system, because pore spaces of sands and gravels form natural reservoirs of much larger capacity than could be obtained in artificial basins. Such storage does not invite evaporation or growth of organisms. Many subsurface and filtered waters cannot be kept in open reservoirs without deterioration.

Subterranean Storage Gallery, Santa Barbara, Cal.* Tunnel, 5 ft. × 7 ft., penetrates 2 miles through sandstone and shale into a hill containing a natural underground reservoir. 2000 ft. from tunnel portal is a masonry bulkhead; through this passes an 8-in. pipe which conveys the underground supply to the city reservoirs during the dry season. A valve outside the bulkhead is closed in autumn when city no longer needs this auxiliary supply; during the winter, the pressure behind the bulkhead rises rapidly. In the spring about 1 mgd. is available for 3 months. The underground reservoir has an estimated capacity of 45 mg. Normal flow from underground is about ½ mgd.

Bibliography. An exhaustive bibliography of American and foreign studies of underground waters appears in Nineteenth Annual Report of the U. S. Geological Survey, 1897–1898, Part II, p. 381; also in Bibliographic Review and Index of Papers Relating to Underground Waters, Published by the U. S. Geological Survey (1879–1904) by Myron L. Fuller, Water Supply & Irrigation Paper, No. 120, 1905.

* E. R., Jan. 22, 1916.

PART II
COLLECTION OF WATER

CHAPTER V

INTAKES

Intakes in Sand. For years the supply of the Citizens Water Company, Denver,* was secured from about a mile of timber crib, 30 in. square in section, and about 1 mi. of perforated 30-in. pipe, both submerged from 14 to 22 ft. in water-bearing sands of Platte river. Timber crib was open at bottom, so that with openings of cribbing perhaps half its superficial area permitted inflow; for the pipe the net area of perforations probably bore a much smaller ratio to circumferential area. It may be assumed that in 2 mi. the total area

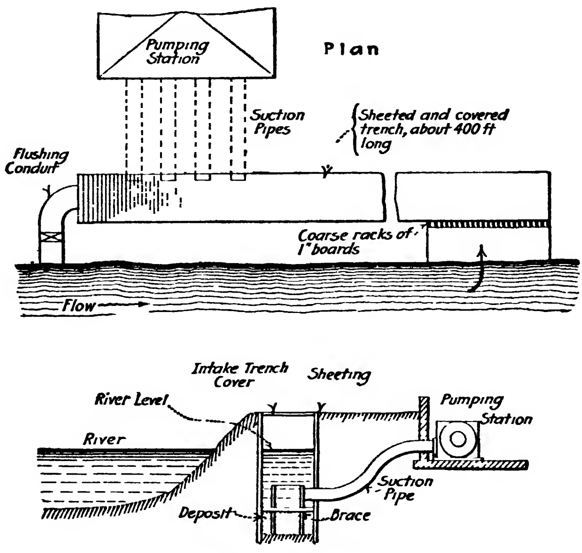


FIG. 26.—(See p. 90.)

of inlet openings equalled 25 per cent. of the surface exposed to sand, affording a total net area of inlet openings of 26,000 sq. ft. Supply secured was 400,000 to 450,000 cu. ft. per day, equivalent to 13 cu. ft. per day per sq. ft. of inlet opening, or velocity of inflow of 0.00015 ft. per sec. As the water approached radially from every direction it must be assumed that the maximum velocity

* Partly from T. A. S. C. E., Vol. 31, 1894, p. 133.

through the sand was less than one-fourth that through the openings. Assuming an average head of 16 ft. acting on the cribs, the spouting velocity of water entering the pipe from a free body of water would be 32 ft. per sec.; actual average velocity was only $\frac{1}{200,000}$ of such rate, corresponding to $V = 0.00004\sqrt{H}$.

A River intake, of excellent type, is shown in Fig. 26. A sheeted and covered trench in the river bank parallel to the flow had cross-sections sufficiently large to give low, depositing velocities, thus permitting grit to settle before reaching the suction pipes. By opening the flushing conduit at suitable times, strong flows were created which removed the sediment.

Intakes on the Great Lakes.* Experience has demonstrated that, not considering pollution, intake pipes of considerable length are required in lakes to maintain draught, owing to the action of ice and the movements of sand, particularly in shallow waters.

Lake Michigan. Depths of water from Waukegan around South Shore, and North on East Shore up to Manistee.

Distance out, ft.	Depth, ft.	Distance out, ft.	Depth, ft.
1,000	15	10,000	60
2,000	30	15,000	72
3,000	40	20,000	120
5,000	45		

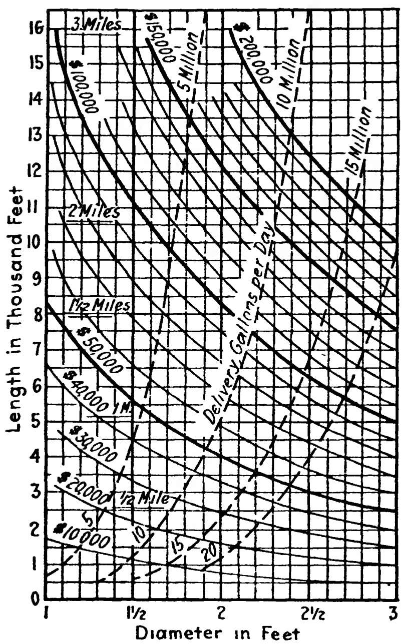


FIG. 27.—Relation of diameter, length, delivery and cost of cast-iron intake pipe.

(20 ft friction loss)

Off-shore storms in shallow water erode the sand bottom rapidly, tending to clog intakes or embarrass pumping operations through wear on the pumps. Ice, driven by up-shore winds, piles in huge windrows, often 10 to 20 ft. above the lake level, in shallow water, filling the lake solidly to the bottom to such an extent as to shut off water supplies drawn close to the shore; 2000 to 3000 ft. off-shore in 20 to 30 ft. of water is required. In larger supplies, particularly at Chicago, much trouble is experienced with anchor ice as far out as 4 mi. in 40 ft. of water; it is largely a matter of total draught in one locality and the velocity of the water as it enters the intake. In depths exceeding 40 ft., the influence of waves is insufficient to cause serious washing and consequent turbidity. The distance of the intake from the shore also has a bearing.

Diagram 27 is believed to cover first-class practice in ordinary circumstances on Lake Michigan, where it is

* Chas B Burdick, Illinois Water Supply Ass'n, Feb. 21, 1911.

necessary to dredge in clay and sand practically throughout the length of intake; cost of crib or shore well is not included. A 7-ft. tunnel, 3 mi. in length, in clay, under favorable circumstances, can be built for about $25 per ft., including shafts, but exclusive of intake crib.

Allowance must be made for deterioration in carrying capacity of the pipes and against the reasonable variations in the rate at which water is demanded. This is particularly important in the smaller supplies.

Table 31. Cost of Cast-iron Intake Pipes

Diam. of pipe, in	Pipe, per ft	Flexible joints		Trenching and laying, per ft	Total cost per ft laid
		Per joint	Per ft		
12	$1.09	$35	$0 36	$4 80	$6 25
16	1.62	52	0 54	5 90	8 06
18	1 94	59	0 61	6 50	9 05
20	2 25	69	0 72	7.10	10 07
24	3 05	92	0 96	8 20	12 21
30	4 38	150	1 56	10 00	15 94
36	5 90	240	2 50	11.50	19.90

Note. Pipe $30 per ton. One flexible joint every 8 lengths. (See also p 403.)

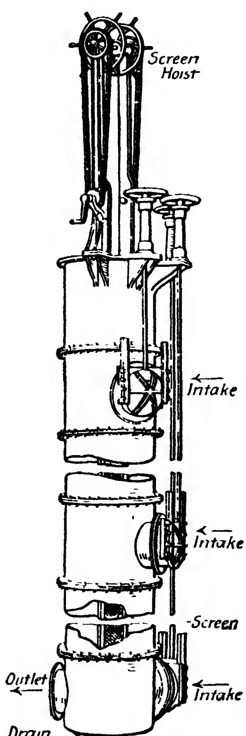


Fig. 28.—Wilcox intake pipe.

Wilcox intake pipe is arranged with sluice-gates, screens, screen-hoists, outlet pipe and a drain valve. The sluice gates are at different elevations so that water can at any time be taken from near the surface, bottom, or an intermediate depth as desired. This prevents to a great extent sediment entering the supply pipe. Screens are in suitable grooves and are attached to the hoisting apparatus so they can be easily cleaned and kept in good order. The drain valve in the bottom provides for blowing out sediment. A small ladder is fastened in place inside. This pipe can be built solidly in concrete, leaving the sluice-gates projecting beyond the face of the masonry. Made by Coldwell-Wilcox Co., Newburgh, N. Y.

Vancouver Intake. The creek carries large quantities of boulders, gravel and silt during freshets; to keep these out of the pipes, the intake was placed so that water entering the screens flows

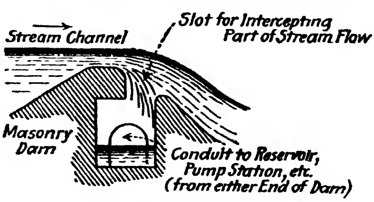


Fig. 29.—Differential intake.
(Konig Wasserleitungen, 1907, p. 257.)

at right angles to the stream and at less velocity. Thus floods carry trash past the intake. (E. R., Dec. 24, 1910.)

Anchor Ice and Frazil are very important in Great Lakes. They are variously termed frazil, ground ghu, lappered ice (Scotland), moutonne and spicular ice. Anchor ice is formed on bed; frazil between bottom and surface, and only when there is open water above. On Lake Michigan, further conditions for formation of anchor ice and frazil are westerly wind and thermometer below freezing; never when sun is shining, or when sky is clouded at night. To clear the intake at Lake Forest waterworks, the valves in the well are closed, thereby throwing any pressure remaining in the mains upon the intake through a by-pass. At times a pressure of 75 lb. has been maintained for 20 min. before there would be noticeable relief. About one out of five times this method, if tried before sunrise, has proved satisfactory; rest of time more water is wasted than comes in through the intake before it is again clogged. (L. C. Trow, Supt. Lake Forest Water Co., Illinois W. S. Ass'n paper, 1911.) Such troubles with ice are experienced on other lakes also, and along the Mississippi and other rivers.

CHAPTER VI

WATERSHED DEVELOPMENT BY RESERVOIRS

Fundamental Considerations. In calculating, for water supply, the safe yield from large watersheds, it is usual practice to provide for a safe uniform draft for a series of years. A large amount of storage which cannot be drawn off in a single year, and which can be exhausted only after the reservoirs have failed to fill in a wet season, represents a better condition than a reservoir holding but one dry year's supply; because (1) two or more dry years are less likely to occur in succession, although more commonly occurring at intervals; and (2) a larger reservoir gives a longer period for action after warning of need of an additional supply. A water famine scare comes long before the reservoir is exhausted to the level assumed in capacity computations. Other things being equal, the storage per sq. mi. should be directly proportional to the yield. Swampy lands detract from the storage value of a watershed, as they aid evaporation.

Economic Development. Points to be considered in studying the economic development of a watershed are: (1) Area tributary to each possible reservoir. (2) Required development of storage in each. (3) Storage-elevation curve for each. (4) Cost-elevation curve for each. (5) Cost-diameter and diameter-gradient-quantity data for all connecting aqueducts. (6) Combined plot of available-storage-cost curves for all reservoirs and their connecting aqueducts. (7) Most economical combination of reservoirs within the watershed, and cost-storage curve for this combination for different percentages of total storage on the watershed.

Safe yield of watershed is necessarily based upon practical considerations of rainfall and run-off, or, if these data are not available, upon like data from a similar watershed, properly weighted for local conditions. Experience and computations in New England have shown that only small storage is necessary to obtain 200,000 to 300,000 gals. daily per sq. mi.; but for larger supplies, say 800,000 to 1,000,000 gals. daily, large storage is required for absolute safety in dry years, little of this storage being utilized in ordinary years. A general rule is that above, say, 500,000 or 600,000 gals. daily per sq mi., each additional 100,000 gals. requires much more additional storage than the preceding 100,000 gals., and a point exists, beyond which, unless storage can be obtained at remarkably low cost, it is not warranted by economics to proceed.

Testing Imperviousness of Reservoir Bottom. To test the site of high-service reservoir, Forest Park, Baltimore, Alfred M. Quick used following methods: Over the ledge of gabbro and serpentine were clay, disintegrated serpentine and top-soil. Dam 5 ft. high, of rammed clay, upstream slope 1 on 2, and downstream face of plank, vertical, was built across stream in reservoir site and subjected to head of water 3 or 4 months, sometimes overtopped. A 2-ft. cubical wooden box was rammed full of the clay, into which a 1-in. pipe with end pointed and perforated was driven 15 in. and put under head of 40 ft.

for 4 days. Both dam and box were rammed by hand. A 4½-ft. piece of 6-in. wrought-iron pipe with capped ends was filled with the clay and another with the disintegrated serpentine, rammed with a jack. About 4 in. of graded gravel was put in bottom of each and lower cap had gage cock tapped in. Through top cap water under 140 to 160 lb. per sq. in. was forced for short time, then under 40 lb. for 6 months. In each test water penetrated not over 1½ in.

IMPOUNDING RESERVOIRS*

Drafts from Reservoirs. Records for a long series of years on Croton, Sudbury and Nashua rivers furnish accurate bases for determination of reser-

Table 32. Storage Capacity Required to Sustain Constant Daily Draft from 1 Sq. Mi. Containing Various Percentages of Water Surface†
(Based on Sudbury Watershed, 1875–1890)

Constant daily draft, hundred thousand gals.	Storage, million gals. per sq mi								
	0%	2%	4%	6%	8%	10%	15%	20%	25%
1 0	0 314	1 289	2 656	6 973	10 992	15 012	26 883	40 224	53.565
1 5	3 006	4 711	7 552	11 573	15 592	19 642	32 983	46 324	59.665
2 0	8 797	9 937	12 802	15 666	20 427	25 742	39 083	52 424	65 765
2 5	17.997	20 637	23 502	26 366	29 230	33 338	45 449	58 524	71 865
3 0	28 473	31.337	34 202	37 066	39 930	43 437	54 599	66 702	78 807
3 5	39.173	42.037	44.902	47 766	50 630	54 137	64 812	75.852	87 957
4 0	51 303	52 788	55 602	58 466	61 643	66 050	77 062	88 076	99 089
4 5	63 553	65 038	66 525	69 488	73 893	78 300	89 312	100 970	127.412
5 0	75 803	77 288	79 105	82 131	86.143	90 550	103 474	129 920	156 362
5 5	88 053	89 877	92 905	95 931	98.958	105 987	132 424	158 870	185 312
6 0	100 651	103.677	106 705	113 781	124 357	134 937	161 374	187 820	214 262
6 5	114 451	121.577	132.154	142.731	153 307	163.887	190 324	216 770	250.744
7 0	139 950	150.527	161 104	171 681	182 257	192 837	219 274	265 546	336 044
7 5	168 900	179.477	190 054	200 631	211 207	221 787	280 343	350 846	421 344
8 0	199 106	208.427	219 004	244 445	270.452	297.460	365 643	436 146	506 644
8 5	250 328	276 275	302 240	328 195	354 202	380 557	450 943	521 446	591 944
9 0	334 078	360 025	385 990	411 945	437.952	465 857	536 243	606 746	677 244

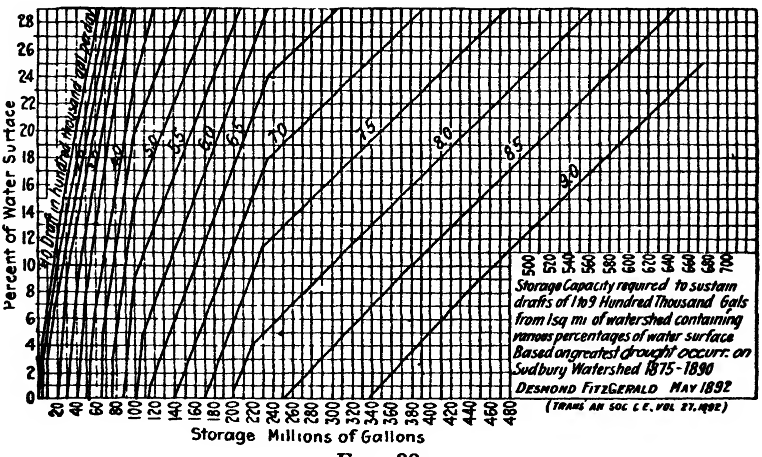


Fig. 30.

voir storage and depletion for their own and similar watersheds. Draft on public reservoirs is practically constant, or increasing in a somewhat definite

* Consult also A. R. Binnie's "Rainfall Reservoirs and Water Supply" (Constable & Co., 1913).
† Reservoir storage, FitzGerald; T A S. C E, Vol 27 1892, p 267.

manner, and is subject to fairly regular seasonal variations, whereas water is received into them with great irregularity. To maintain a supply at the desired rate, it is necessary to establish certain general relations between draft and reservoir storage. The following tables reflect differences of judgment and method. See also page 114.

Table 33. Storage Capacity Required to Make Available Different Daily Volumes of Water per Sq. Mi. of Watershed (Estimating Land Surfaces only) Corrected for Effect of Evaporation and Rainfall on Varying Percentages of Water Surface (not Included in Estimating Area of Watershed)*

Constant daily draft, hundred thousand gals.	Storage required, in million gals. per sq. mi. of land surface, to prevent deficiency in season of greatest drought, with following percentages of water surface				
	0%	3%	6%	10%	25%
1 0	0 556	3 0	8 8		
1 5	3 4	7 1	13 4		
2 0	9 4	11 7	18 0		
2 5	19 0	22 2	25 4		
3 0	29 8	33 0	36 1		
4 0	52 0	54 4	57 5		
5 0	76 5	77 3	80 3		
6 0	102 0	104 6	107 1	112 8	
7 0	144 4	153 0	161 6	170 7	215 9
8 0	202 3	210 9	219 5	228 6	273 8
9 0	346 2	349 2	352 2	353 9	381 6
10 0	514 6	516 7	519 7	523 6	532 2

Table 34 and Fig. 31 are also for computing safe capacities of sources of supply. To apply practically the table or the diagram showing storage capacity required to supply different daily drafts of water from 1 sq. mi. of drainage area, it is necessary to have the following preliminary information:

(1) Drainage area in square miles. (2) Area of all water surfaces when reservoirs are full, in square miles, together with 40 per cent. of area of undrained swamps and 30 per cent. of area of drained swamps. By dividing total area of water surfaces so determined by number of square miles in drainage area, per cent. of water surfaces when reservoirs are full is obtained. (3) *Available* capacity of storage reservoir, in million gallons. *Available* capacity of storage reservoirs divided by number of square miles in whole drainage area gives storage per square mile.

Size of Impounding or Storage Reservoirs. The primary purpose of a storage reservoir is the conservation of the water which falls upon a watershed; the larger it is, within practical limits, the greater is the proportion of the flood water of any series of years which can be made available. As the size of the reservoir is increased beyond moderate limits, a greater proportionate increase in the quantity of storage is required for each additional million gals. per day which such storage will furnish, and it is generally found in-

* Reservoir Storage, F P Stearns, E. N., Nov. 7, 1891. This table excludes water surfaces from the computed gathering area as having no value for collection purposes, and makes a correction for the balance between evaporation and rainfall on the surfaces of the bodies of water into which the land areas included in the computations drain. Like Table 32, this one is based on Sudbury data.

expedient to attempt to secure from a watershed more than 80 per cent. of the average run-off. Notwithstanding this general rule, there are exceptional cases in which a larger draft may be obtained at moderate cost.

Table 34. Increment in Storage Capacity Corresponding to Increment in Daily Supply from 1 Sq. Mi. of Drainage Area Containing Various Percentages of Water Surface (Based on Fig. 31)*

Increment in daily supply, gallons per sq. mi.	Increment in required storage. Million gallons			
	0%	5%	10%	15%
200,000 to 250,000	7.2	7.2	7.6	7.6
250,000 to 300,000	7.8	7.9	8.2	8.2
300,000 to 350,000	8.4	8.6	8.8	8.8
350,000 to 400,000	9.0	9.3	9.4	9.4
400,000 to 450,000	9.6	10.0	10.0	10.1
450,000 to 500,000	10.2	10.7	10.7	11.0
500,000 to 550,000	10.9	11.5	11.9	12.4
550,000 to 600,000	11.7	12.7	13.6	14.6
600,000 to 650,000	12.9	14.3	15.8	17.6
650,000 to 700,000	14.4	16.3	18.5	21.6
700,000 to 750,000	16.3	18.7	22.0	26.6
750,000 to 800,000	18.8	21.7	26.0	33.0
800,000 to 850,000	22.6	26.5	33.0	43.0
850,000 to 900,000	28.8	35.0	45.0	57.0
900,000 to 950,000	38.7	49.5	62.0	75.0
950,000 to 1,000,000	50.6	71.0	83.0	.
1,000,000 to 1,050,000	74.8	103.0		

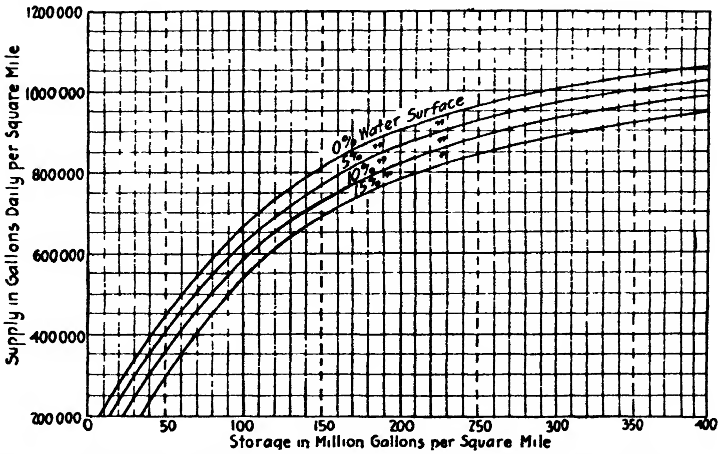


FIG. 31.—Storage capacity required to supply various quantities of water daily from one square mile of drainage area containing various percentages of water surfaces.*

Composite diagram based on Abbott Run (R. I.), Sudbury and Manhan rivers, Tillotson Brook Wachusett reservoir (Mass.), Naugatuck river (Conn.), Croton river (N. Y.). See Table 34 and Fig. 13.

Impounding reservoirs should be proportioned so that a constant draft in dry years will not expose the margins below high-water mark for longer than 2 yrs.; otherwise vegetation gets a strong start, and clearing is necessary;

*Jl. N. E. W. W. A., 1914.

vegetation injures quality of water and clearing is expensive. A reservoir which cannot be emptied by the draft in less than 3, 4, or 5 yrs., allows for sufficient warning against the possible shortage. Full utilization of an available watershed calls for the largest feasible development.

On "Storage to be Provided in Impounding Reservoir for Municipal Water Supply," see paper by Allen Hazen in Trans. Am. Soc., C. E., Vol. 77, 1914; which contains new methods of analysis, diagrams and formulas.

One Larger vs. Several Small Reservoirs. Even if a number of small reservoirs give an economy in first cost over one large reservoir, they are not advisable for the following reasons: Small and shallow reservoirs are easily affected by organic growths, producing undesirable tastes and odors. (2) They are sensitive to the influence of vegetation, swampy areas, etc. (3) Water passes through too quickly, giving no time for sterilizing action. Most pathogenic bacteria will die after 2 to 4 weeks in a large storage reservoir.* (4) With a large, deep reservoir water can be drawn from the surface, bottom or intermediate depths, thus securing good water from one of these levels, when others are unsatisfactory. (5) The quality of the water is improved by sedimentation and bleaching. (6) The greater depths of a large reservoir, and the greater exposure to the wind, tend to prevent growths of objectionable organisms. On the other hand, the quality may be injured if the reservoir is so large as to be filled only at times of excessively heavy run-off. During dry spells, injurious substances are liable to collect on the margins.

Preventing Overflow from Watersheds. Generally the cost is disproportionate to the gain in supply. Studies may be made to determine the additional reservoir capacity required to prevent any flow over the spillway of the only or the lowest reservoir on the watershed as follows: To find the average supply available, plot a mass curve of run-off from the longest records available, supplementing them if desirable by methods explained elsewhere (see p. 3). The average supply obtainable, if waste is avoided, may be found by drawing a straight line between the ends of this curve. A line parallel to this line and tangent to the highest point of the mass curve is next constructed. The maximum ordinate between the mass curve and this second line gives the maximum storage required, if this daily rate is to be continued.

Leakage. Besides danger involved in many cases by percolation beneath dams, loss of water from the reservoir is often a serious consideration, and objectionable wetness may be caused in places near the reservoir, leading to claims for damage. Some sites, otherwise suitable, cannot be used for reservoirs because the perviousness of the bottom and sides is so great that objectionable seepage would ensue. Extreme cases are on record wherein reservoirs constructed on such sites have been abandoned because they could not hold water, without a complete and prohibitively expensive, water-tight lining.

SPILLWAYS FOR RESERVOIRS AND CANALS

Capacity. Waste weir and spillway capacities are governed by rainfall and run-off. In eastern United States, a waste weir and its channel should be of sufficient dimensions to carry off a 6-in. rainfall on the watershed in 24 hrs.

*See also p 681.

Cloudbursts seldom cover an area greater than 40 sq. mi. Spillways of small reservoirs must be relatively larger than for large reservoirs. Spillways of reservoirs on small watersheds should be relatively larger than for a reservoir of corresponding size on large watersheds. Spillways for some important reservoirs were designed to discharge 8 in. of run-off per 24 hrs. Spillways of Scioto dam at Columbus, and of reservoirs of the Croton and Sudbury supplies were designed for 6-in. run-off; those of Wachusett reservoir for Boston and Catskill reservoirs for New York, for 8-in. run-off. Other examples could be given.

Spillways with Steep Slope. In determining widths of spillway at successive sections for a Yuba River débris barrier (Cal.), the depth was assumed constant and velocity, V_s, was calculated by the following formula ("Velocity of Water Flowing down a Steep Slope," E. P. Hill, in Proceedings, Inst. C. E., Vol. 161 (1904–05), p. 345), somewhat simplified:

$$V_s = \left[V^2 + \frac{1}{\epsilon^{(V_v - V)^2}} (V^2 - V_a^2) \right]^{\frac{1}{2}}$$

where $V = C\sqrt{RS}$, Chezy's formula for velocity in watercourses, ordinary slopes (see p. 265).

$V_v = \sqrt{2gH}$ = velocity, due to gravity, of body falling freely in vacuum

V_a = velocity of approach

ϵ = base of Napierian logarithms = 2.718.

This formula is based on two assumptions: that experimental values of C in Chezy's formula are applicable to high velocities; and that, within narrow limits, the hydraulic mean radius may be taken as constant at its mean value. Velocity of water in a channel on a steep slope will increase until acceleration is counteracted by frictional resistance, when $V_s = V_a$, and the formula reduces to $V_s = V = C\sqrt{RS}$, as it should. That this formula is not limited to slopes of small degree is shown by the fact that for high values of S, as for instance $S = 0.707$ (45° slope) or $S = 1$ (vertical drop), substituting numerical values for H and R, reduces the expression to the form $V_s = \sqrt{a + bV_a^2}$, in which a is but little less than $2gH$ and b is almost unity; that is the value of V_s is very little less than $\sqrt{2gH + V_a^2}$, which would be its value were it influenced by gravity alone. (H. H. Wadsworth, in T. A. S. C. E., Vol. 71, 1911.)

Siphon Spillways. The siphon spillway, patented by G. F. Stickney, is a closed conduit of an inverted U shape, extending through a dam (where a fall is available), which utilizes the siphonic principle to induce a high velocity of flow. Action is entirely automatic and the flow, when established, is comparable to that through a submerged sluice gate, with a velocity from 60 to 80 per cent. of the theoretical velocity due to the head. The crown of the siphon is entirely above low-water level. The upstream leg is sufficiently long to bring the inlet well below the water surface so as to avoid the entrance of drift, ice, etc. The downstream leg is made as long as practicable in order to take advantage of all the head available. The outlet need not be submerged, and in many cases should be open to the air. Action is controlled by an air

vent which pierces the upstream wall at low-water level. When the water above the dam rises, the vent is submerged, so that no air can pass into the siphon, and the water spills through the crown and the downstream leg. At a certain depth of overflow, the air in the siphon begins to pass out with the flowing water, and, as the air becomes rarified, the water rises to a higher level in the crown than the surface of the stream outside. Within a short period, the crown becomes completely filled with water, the siphon is primed and rapid flow is established which will continue until the water surface is drawn down so as to expose the vent. A moderate amount of air can be admitted to the siphon while in operation, without producing other effect than decreased efficiency. A fluctuation of water surface of 3 in. to 1 ft., depending on size of siphon, is necessary to start and to stop the flow. Siphon of large capacity may be built with crown well above the highest water level, but, in such a case, a simple automatic priming device is necessary* and both ends of the siphon must be submerged, Fig. 32. The conduit is short and flow rapid, under which conditions no air can collect.

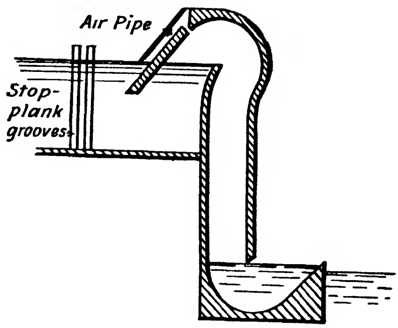


Fig. 32.

On the N. Y. Barge canal the ratio of length to that of an ordinary spillway varies from 1 to 3 to 1 to 5, depending on the available head. Economy in cost of construction is considerable. The spillway at Lock No. 9 receives a maximum inflow of 700 cu. ft. per sec. It was desired to provide for this flow and limit fluctuation of water-surface to about 1 ft. An ordinary waste-weir with a depth of only 1 ft. would require a spillway 200 ft. long. The siphon spillway, however, measures but 57 ft. between abutments. This structure consists of 4 siphons and a waste-weir 20 ft. long, the purpose of the weir being to carry off floating débris. Each siphon has an area of 7.75 sq. ft. Acting under a head of 10.5 ft. each will discharge approximately 160 cu. ft. per sec.; while the waste-weir, with a depth of 1 ft., will discharge 70 cu. ft. per sec. The inlet of the siphon is placed well below the water surface, and is protected by a screen, to prevent the entrance of floating bodies. The inlet is flared to double the normal area, to reduce entry loss. As the siphon will not act until completely filled, it is necessary to limit the hight at crown to 1 ft., but the necessary area was obtained at this point by increasing the width. From the crown to the outlet, the siphon is practically uniform in area, although the dimensions were changed from 1 × 7.75 to 2 × 4 ft., to give a better section to the masonry and facilitate removal of forms. At the crown, where removal of forms would have been difficult, iron castings were provided. Three vents, each 6 in. high by 12 in. wide, pierce the wall at low-water level, above the inlet

* "By various simple adjustments, a series of such spillways can be so arranged as to prime themselves automatically at different levels, and thus secure any desired gradation in rate of discharge" From "Dams," Davis & Henny, International Engineering Congress, 1915.

of each siphon. When the water has been drawn down to this level, air will enter through these vents, and stop flow through the siphon. A little below these vents, a single opening of the same size acts as a precautionary vent to break the flow in case the upper openings become clogged in any way. (E. N., Oct. 13, 1910.)

Tests of siphonic spillway at the flume of the Tennessee Power Co., Ocoee river, Tennessee (used to conduct away safely the waters of the flume, carrying 1200 cfs., when turbine gates are suddenly closed), gave value of $C = 0.62$ to 0.65, in formula: $Q = AC\sqrt{2gH}$. Q during test of 2 siphons = 423 cfs.

Head, $H = 25.5$ ft. Area of each siphon = 8 sq. ft.; $A = 16$. Speed of priming was also determined. Plant described in E. R., Apr. 18, 1914, p. 454. (W. P. Creager, E. R., May 16, 1914.)

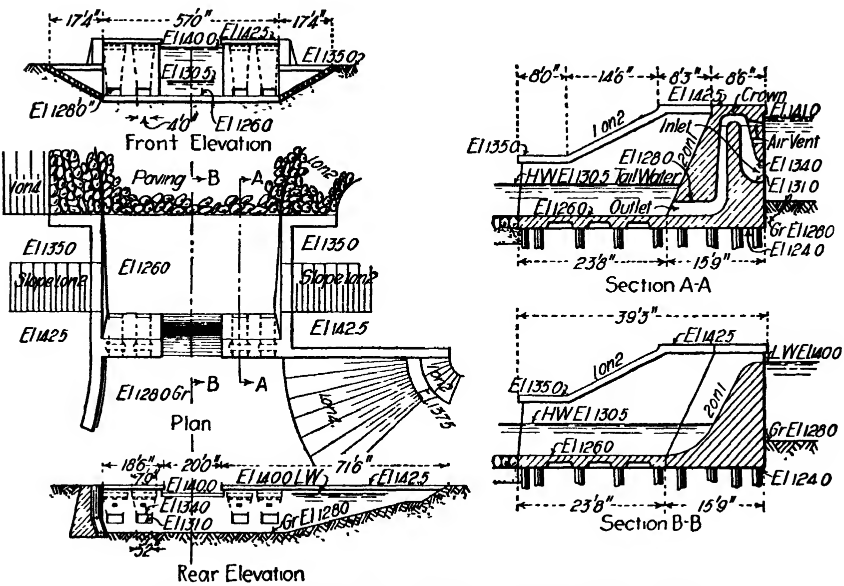


FIG. 33.—Siphon spillway, Lock No. 9, N. Y. Barge Canal.

Waste Weir and Rise of Reservoir.* The following equation gives rise of surface of reservoir under flood conditions (prismatic reservoir, uniform inflow)

$$t = \frac{A(H_2 - H_1)}{I - \frac{Q_1 + Q_2}{2}}$$

t = time in seconds for the surface to rise from H_1 to H_2 as observed on the weir.
H_1 = depth in feet on weir at beginning of flood flow.
A = area of reservoir, square feet, at weir level.
I = rate of inflow, cubic feet per sec.

* U. S. Geological Survey, Water Supply Paper No. 150, 1906.

Q_1 = outflow, cubic feet per sec., corresponding to H_1.
Q_2 = outflow, cubic feet per sec., corresponding to H_2.

Table 35. Temporary Storage above Waste Weir Crest, Surface Area of Reservoir, Equivalent Quantity Collected, or Run-off, from Watershed in 24 Hrs., and Length of Waste Weir for Three Assumed Reservoirs and Watersheds, D, E, F.

Assumptions.			*D*	*E*	*F*
Area of reservoir, at weir crest level	sq. ft.,	=	1,000,000	10,000,000	100,000,000
	acres	=	23	230	2,300
Area of watershed, sq. mi.		=	1	10	100
One inch collected from watershed in 24 hrs. is equivalent to run-off of 26.888 cfs per sq mi.		=	27	270	2,700
One inch depth on watershed,	cu. ft.	=	2,323,000	23,232,000	232,320,000
	gals.	=	17,379,000	173,787,000	1,737,870,000
Length of shore line, or perimeter of reservoir, at crest level = $10\sqrt{A}$	ft.		10,000	31,600	100,000
	mi.		1.9	6.0	19

Average slope of shores 1 vertical on 5 horizontal

A = area of reservoir at level of waste weir crest.
p = perimeter of reservoir at level of waste weir crest.
H = hight, or rise, above level of waste weir crest.
G = gain in storage above level of waste weir crest.

Gain in storage, $G = AH + C_wH^2p$ (very nearly) (1)

C_w is a coefficient depending upon slope of shore, and in case assumed = 2.5 (*i.e.*, $\frac{1}{2}$ base, or horizontal, of slope ratio, 1 on 5).

Area of water surface for any value of H is:

$A_s = A + 2C_wHp$ (very nearly) (2)

For weir length, broad crest, Q = run-off, cfs. (after steady conditions have been attained), $= 3.40\,LH^{\frac{3}{2}}$ nearly enough; whence weir length, in feet,

$L = Q \div 3.40H^{\frac{3}{2}}$ (3)

Formulas, (1), (2), (3), Tables 35 and 36, and Figs. 34–36, are useful for determining storage gained by flashboards or by raising a waste weir or dam; run-off required from watershed to cause such gain; quantity going into temporary storage during a freshet while the head increases on the waste weir; also length of weir for given head H and corresponding assumed conditions.

Economic Length of Waste Weir. For a given inflow into a reservoir the flood level is a function of the length of the weir. The top elevation of the dam having not yet been fixed economically, and assuming the top of the dam to be at a constant distance above flood level, the inflow provided for will vary with assumed flood level, and consequently with the length of weir. The economical weir length will therefore be the one corresponding to the minimum point on the curve of "hight of dam: total cost of reservoir." In plotting this curve, "cost of reservoir" should be plotted as ordinates, that is, to a vertical

of each siphon. When the water has been drawn down to this level, air will enter through these vents, and stop flow through the siphon. A little below these vents, a single opening of the same size acts as a precautionary vent to break the flow in case the upper openings become clogged in any way. (E. N., Oct. 13, 1910.)

Tests of siphonic spillway at the flume of the Tennessee Power Co., Ocoee river, Tennessee (used to conduct away safely the waters of the flume, carrying 1200 cfs., when turbine gates are suddenly closed), gave value of $C = 0.62$ to 0.65, in formula: $Q = AC\sqrt{2gH}$. Q during test of 2 siphons = 423 cfs.

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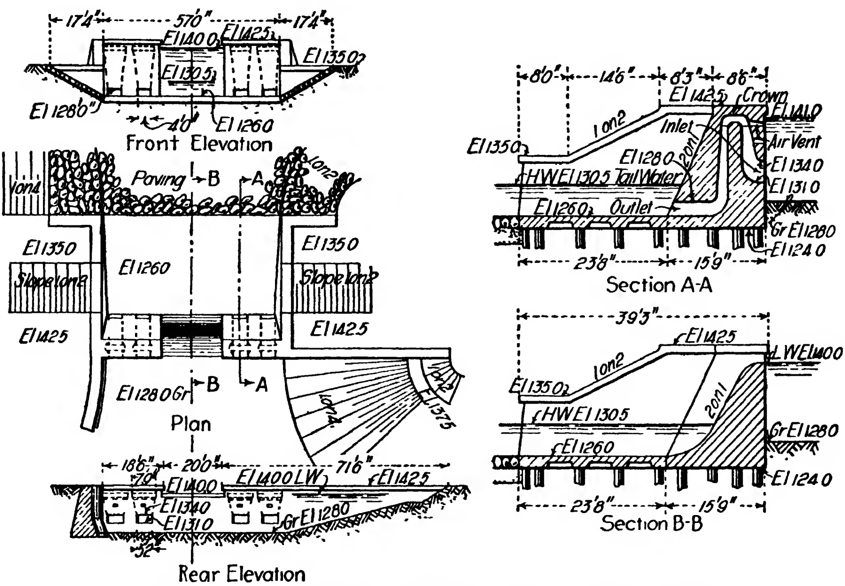

Fig. 33.—Siphon spillway, Lock No. 9, N. Y. Barge Canal.

Waste Weir and Rise of Reservoir.* The following equation gives rise of surface of reservoir under flood conditions (prismatic reservoir, uniform inflow)

$$t = \frac{A(H_2 - H_1)}{I - \dfrac{Q_1 + Q_2}{2}}$$

t = time in seconds for the surface to rise from H_1 to H_2 as observed on the weir.

H_1 = depth in feet on weir at beginning of flood flow.

A = area of reservoir, square feet, at weir level.

I = rate of inflow, cubic feet per sec.

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One inch depth on watershed, cu. ft		=	2,323,000	23,232,000	232,320,000
	gals.	=	17,379,000	173,787,000	1,737,870,000
Length of shore line, or perimeter of reservoir, at crest level $= 10\sqrt{A}$	ft.		10,000	31,600	100,000
	mi.		1 9	6 0	19

Average slope of shores 1 vertical on 5 horizontal

A = area of reservoir at level of waste weir crest.
p = perimeter of reservoir at level of waste weir crest.
H = hight, or rise, above level of waste weir crest.
G = gain in storage above level of waste weir crest.

Gain in storage, $G = AH + C_w H^2 p$ (very nearly) (1)

C_w is a coefficient depending upon slope of shore, and in case assumed = 2.5 (*i.e.*, ½ base, or horizontal, of slope ratio, 1 on 5).

Area of water surface for any value of H is:

$A_s = A + 2C_w Hp$ (very nearly) (2)

For weir length, broad crest, Q = run-off, cfs. (after steady conditions have been attained), $= 3\ 40\ LH^{\frac{3}{2}}$ nearly enough; whence weir length, in feet,

$L = Q \div 3.40H^{\frac{3}{2}}$ (3)

Formulas, (1), (2), (3), Tables 35 and 36, and Figs. 34–36, are useful for determining storage gained by flashboards or by raising a waste weir or dam; run-off required from watershed to cause such gain; quantity going into temporary storage during a freshet while the head increases on the waste weir; also length of weir for given head H and corresponding assumed conditions.

Economic Length of Waste Weir. For a given inflow into a reservoir the flood level is a function of the length of the weir. The top elevation of the dam having not yet been fixed economically, and assuming the top of the dam to be at a constant distance above flood level, the inflow provided for will vary with assumed flood level, and consequently with the length of weir. The economical weir length will therefore be the one corresponding to the minimum point on the curve of "hight of dam: total cost of reservoir." In plotting this curve, "cost of reservoir" should be plotted as ordinates, that is, to a vertical

Table 36. Temporary Storage Above Waste Weir Crest, Area of Water Surface, Corresponding Run-off and Length of Weir

H ft	G million cu ft.	A_s million sq ft	Inches depth on water-shed per 24 hrs.	Run-off, c.f.s. Q	Waste weir, length, ft.
		Reservoir D. Area 1,000,000 sq ft. (23 acres)			
0 05	0 050	1.002	0.021	0.56	14.8
0 10	0.100	1.005	0 043	1 15	10 9
0 15	0.151	1 007	0.066	1 77	9.1
0 20	0 201	1 010	0.086	2 31	7.7
0 25	0 252	1 013	0 108	2 91	6 8
0 35	0 353	1 017	0.15	4 03	5 7
		Reservoir E. Area 10,000,000 sq ft. (230 acres)			
1	10.079	10.188	0 4	11.6	34
2	20.316	10.376	0 9	23 5	24
3	30.711	10 564	1 3	35 6	20
4	41.264	10.752	1 8	47 7	18
5	51.975	10.940	2 2	60 1	16
6	62.844	11.128	2.7	72 8	15
7	73.871	11 316	3.2	85 7	14
8	85.056	11.504	3 7	98 7	13
9	96 399	11.692	4.2	111 8	12
10	107.900	11.880	4 6	125 0	12
11	119.559	12.068	5 1	138 5	11
12	131.376	12 256	5.7	152 3	11
13	143 351	12.444	6.2	166 0	10
14	155.484	12.632	6.7	180 5	10
		Reservoir F Area 100,000,000 sq ft (2295 acres)			
1	100 250	100.500	0 4	1,160	342
2	201.000	101 000	0 9	2,320	242
3	302.250	101 500	1 3	3,500	198
4	404 000	102 000	1 7	4,690	172
5	506 250	202.500	2 2	5,880	155
6	609 000	103 000	2 6	7,060	141
7	712.250	103.500	3 1	8,240	131
8	816 000	104.000	3 5	9,470	123
9	920 250	104 500	4 0	10,680	116
10	1,025.000	105 000	4 4	11,880	110
11	1,130.250	105 500	4 9	13,100	105
12	1,236.000	106 000	5 3	14,300	101
13	1,342.250	106.500	5 8	15,550	97
14	1,449 000	107.000	6 2	16,800	94

For weir tables, see page 598.

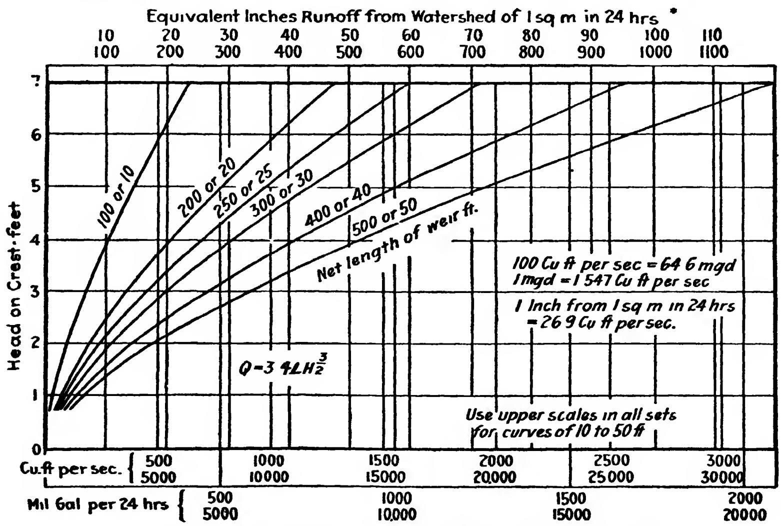


FIG. 34.—Discharge of waste weirs of various lengths.

* Divide by area of watershed in square miles.

scale, and "length of weir" as abscissas. Cost of reservoir includes cost of dam, land takings, treatment of bottom, gatehouses and spillway.

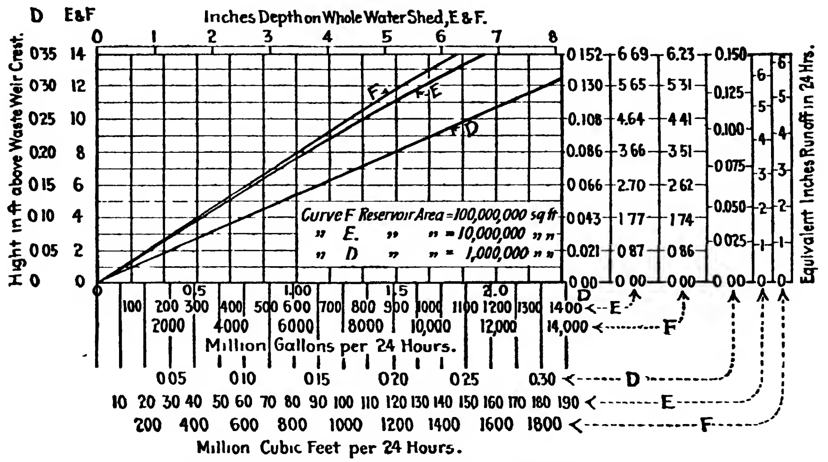


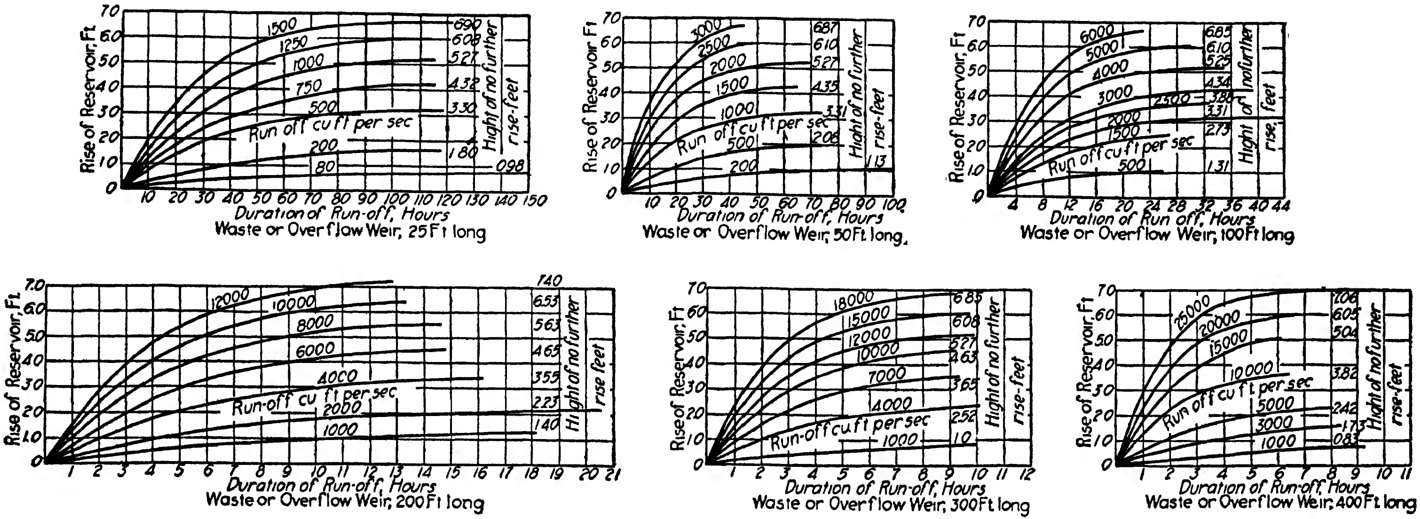
FIG. 35.—Quantity collected, or run-off, from watershed in 24 hours.

CLEARING, GRUBBING AND STRIPPING RESERVOIR SITES

Specifications for Cedar Grove Reservoir, Newark, N. J. Payment was made per acre, acreage being established and shown on the contract drawings before the contract was let. Isolated trees beyond the established lines were included in the excavation price. Incombustible débris was finally buried in borrow pits and acceptably covered, without extra pay. Certain trees were saved for future use in landscaping.

Specifications for Sudbury Reservoir, near Boston, Mass. Muck within embankment lines had to be excavated to clean sand or gravel. Stone walls had to be overturned and muck beneath removed. Bowlder-strewn areas were treated as follows: Bowlders not deeply embedded were rolled aside and mud, muck or loam removed. Measurement of such work included the volume of bowlder below the original surface as earth excavation, but no allowance was made for volume of rock above ground. Large firmly embedded bowlders were left in place; others were deposited in embankment. Débris was promptly burned.

Wachusett Reservoir, Metropolitan Water Supply, Boston. *Clearing and Grubbing.* Clear the sites of all excavations and embankments, and such additional areas adjacent to the sites of excavations and embankments as required and grub the sites of all excavations and such other areas as required. On areas which are cleared and not grubbed, all stumps shall be cut even with the surface of the ground. If required to grub areas where no excavation is to be made, remove all stumps and all roots ½ in. or more in diam. to a depth of 8 in. below the surface of the ground. Trees, brush, stumps and other wood removed shall be burned. For convenience in estimating the area cleared and grubbed, the ground will be divided into squares 25 ft. on a side; only those squares containing two or more stumps, 2 in. or more in diam. measured 6 in.



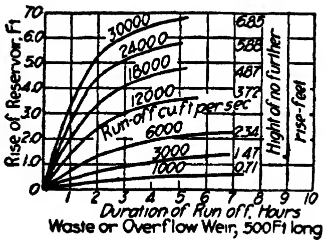


FIG. 36.—Impounding reservoirs, relation of run-off, rise and waste or overflow.

Formula and Notes

$$t = \frac{5163M}{ab^2}\left[1\ 151 \log\left(1 + \frac{3abh^{1/2}}{(a - bh^{1/2})^2}\right) + 0\ 9069 - 0\ 0302 \tan^{-1}\frac{2bh^{1/2} + a}{1\ 732a}\right]$$

t = duration of run-off hours

a = $\sqrt[3]{\text{run-off, cu ft per sec}}$

b = $\sqrt{\sqrt[3]{3\ 33L}}$

L = weir length, feet.

h = rise of reservoir above weir crest, feet.

M = sq mi of water surface at crest level

Formula is in such shape that angle function is read directly from trigonometric tables (i e , not in π measure).

Run-off, as here used, means strictly the quantity of water entering the reservoir,—evaporation and seepage outward being disregarded.

Hight of no further rise is the head on the weir which causes a discharge just equal to the run-off, or inflow

These diagrams apply to 1 sq mi of water surface, and err, on side of safety, in assuming no increase of area with rise. For any other area, multiply duration of run-off in hours by area in sq mi

Duration of run-off is measured from time reservoir level reaches weir crest. Rate of run-off is assumed constant

above the surface of the ground or at a lower level if the engineer shall find it necessary, will be included. Where clearing is to be done and no grubbing, only one-sixth of the area cleared will be measured for payment as clearing and grubbing.

Soil Excavation. Soil shall be excavated to such depths as the engineer may direct. Where the limit of soil excavation is at or near the edge of the reservoir, the soil shall be excavated not only to the elevation of the water when the reservoir is full, but as far beyond this limit as the engineer shall think necessary or desirable in order to prevent the soil from falling into the reservoir in the future, when the shores are eroded by wave action. If excavation to the depth directed does not remove all stumps and all roots more than 1 in. in diam., the contractor shall, without extra charge, do the additional work required to remove the stumps and such roots to a depth of 8 in. below the bottom of the soil excavation. Stone walls and bowlders and stones encountered in excavating soil from the reservoir, unless deeply embedded in the ground, are to be turned over to one side so as to permit the thorough removal of the soil. In the bottoms of kettle holes, or elsewhere, the contractor shall, if required, cover the soil to such depth as the engineer may direct with earth excavated from beneath the surrounding soil excavation. (Criterion for depth of stripping elsewhere was that soil containing more than 4 per cent. of organic matter should be removed.)

Cross River Reservoir, Croton Supply, N. Y. City. Clearing the Reservoir. Clear all lands appertaining to the reservoir below the contour 10 ft. above flow line, of all trees, bushes, logs, stumps, high grass and weeds, and burn these materials or remove them beyond the limits of the grounds owned by the city. Also burn or remove all buildings, wooden fences and other structures designated. All trees 8 in. or more in diam. 3 ft. above ground, shall be sawn very close to the ground; smaller trees shall be chopped so as to leave stumps not over 6 in. high, unless otherwise permitted. Decaying stumps shall be grubbed out to the extent required. Excavate from within and around privy vaults, cess-pools, barnyards, poultry yards and similar places, and dispose of these excavated materials outside the limits of the reservoir; disinfect such materials if and as required.

Ashokan Reservoir, Catskill Water Supply, N. Y. City. *Clearing and Grubbing.* Clear all areas generally of all perishable and other objectionable material, including stumps and roots from the ground ordered grubbed, and leave all areas so far as possible free from perishable material at the time of flooding. As herein used "perishable material" shall include buildings or portions thereof of wood, other wooden structures, boards, fences, trees, brush, bushes, vines, shrubs, logs, stumps, roots, grass, weeds, leaves, sawdust, poles, bridges, rubbish, and other organic matter above the surface of the ground, but not sod nor top-soil, although portions of sod and soil may be removed incidentally in connection with other materials. Within the areas to be grubbed all stumps shall be removed, and all roots more than 2 in. in diam. which are within 6 in. of the surface of the ground. Wherever practicable stumps shall be removed by stump-pulling machines or other similar devices. As stumps, shall be classified the stumps of all trees of whatsoever size, the stumps of all

bushes and shrubs, and the stumps of all vines having a woody fiber. Within the areas ordered grubbed, the stones of all stone walls and stone fences shall be scattered. (Holt caterpillar tractors were used; see Cassier's Magazine, July, 1913.)

Within the areas ordered to be cleared all vegetation shall be cut off. Except where a modification is permitted on those areas which are ordered for subsequent grubbing, the hights at which vegetation shall be cut above the ground are as follows: All trees having a diam. of 12 in. or more, 12 in. from the mean surface of the ground, shall be cut off so that the stumps are not more than 12 in. above the mean surface of the ground. All smaller trees and large bushes shall be cut at a hight not exceeding 6 in. above the mean surface of the ground. All other vegetation shall be cut close to the ground. All existing stumps not cut under this contract, decayed or decaying stumps mentioned below, standing more than 6 in. above the hights specified shall be cut to hights required for trees. All decayed or decaying stumps, where the decayed portion approximates 50 per cent. of the total original cross-section of the stump, shall be broken off and entirely removed.

Where grubbing is ordered on swamp areas drainage ditches will be ordered if necessary. Such ditches will be paid for. On land which has been cultivated and on which the principal growth of vegetation is grass or weeds, the time of clearing may be ordered postponed until immediately prior to flooding. All perishable material cut or removed or found on the ground or caused by this work, shall be completely burned, or otherwise satisfactorily disposed of outside of the city land. The masonry portion of any building shall be thrown down and component parts scattered.

Removing Excrementitious Material. Objectional excrementitious material will be ordered removed from privy vaults, barnyards and stables. Such material shall be buried or otherwise satisfactorily disposed of downstream from the dam or at other acceptable places outside the watershed. After removal of objectionable material, the excavations shall be thoroughly disinfected by application of hypochlorite of calcium. All objectionable material ordered removed shall be transported in vehicles having tight bodies which will not permit scattering of the contents.

The following *unusual points* are also included in these specifications: The entire area reached by the waves is to be cleared. A marginal strip extending vertically from 15 ft. below the normal flow-line to 5 ft. above is to be grubbed as well as cleared. The object of clearing and grubbing is to free flooded areas of any matter which might make the water objectionable. Horizontal areas are used in estimating for payment.

Table 37. Bids Received in 1912 for Clearing and Grubbing Ashokan Reservoir

Description	Unit	Quantity	Lowest bidder	Average of 5 bids
Clearing	Acre	9,500	$30 00	$36.15
Grubbing	Acre	2,400	60.00	86.60
Excavation	Cu. yd.	40,000	0.30	0 70
Plowing and seeding	Acre	150	20 00	36 20
Ordered second cutting of cleared areas.	Acre	400	5.00	13.00
Removing excrementitious material from vaults, yards and stables.	Cu. yd.	1,000	5.00	7.00

Timber and much other salvable material became the property of the contractor, by the terms of the contract.

Stumps. Stump pullers are cheaper than blasting, generally. Caterpillar engines are useful on large jobs. Stumps can be removed by (1) hand, (2 men); (2) burning; (3) blasting; (4) partly excavating in the fall, leaving for the frost to heave out; (5) stump pullers; (6) pull over a tree or large stump with tackle, instead of cutting close and leaving low stump to remove.

Floating Bottom. Peat bogs and the mat of vegetation in swamps tend to float to the surface after being submerged for some time, and often carry stumps with them. This has occurred in a number of places. Stumps tend to become loosened after long submergence. They have little effect on algæ growth; their removal is principally a question of appearance. Floating islands have been found in this way in Goose Creek reservoir, Charleston, S. C., for example.

Effect on Organisms.* No ready and reliable means are known by which prolific organic growths can be wholly prevented in large impounding reservoirs. Copper sulphate is useful, but not to be entirely relied on. Stripping largely reduces one principal source of food for organisms, but there are other sources for every required element of food. Much food can be obtained from the air. Investigation of stripped reservoirs did not reveal one that produced at all times water of desired clearness. Stripping alone is not sufficient; the improvement is not permanent. (Allen Hazen; G. W. Fuller.)

Effect on Tastes and Odors. From a study in 1889 made for Mass. State Board of Health, F. P. Stearns concluded that bad tastes and odors would be most likely to occur in waters stored in unstripped reservoirs, or in waters which received either sewage or purified effluent of sewage; in other words, bad odors and tastes were found to result from providing nitrogenous food for organisms. No question arises as to the much better quality of water in a recently filled stripped reservoir, but doubt exists as to how long this effect will last. There are numerous instances of unstripped reservoirs giving trouble for many years. With an unstripped reservoir, there will certainly be offensive products of decomposition in early years, and a larger growth of organisms, with resulting odors and tastes for an uncertain period. This period, other things being equal, probably has some relation to the frequency with which the water is renewed as reservoirs seldom give trouble when water is stored for only 1 or 2 months. If stripped reservoirs deteriorate, the impression is that the organisms are not such as to cause disagreeable odors or tastes. Waban Hill reservoir, Newton, Mass., containing 15 million gals., has given much trouble from organic growths, bad odors and tastes, although the water is changed an average of once per week. This reservoir was paved with stone.

Results. Hopkinton reservoir, Boston, was substantially as clean and free from organic matter in 1908 as when first stripped in 1888. In Wachusett reservoir bottom, stripped areas not promptly filled with water, were overgrown with weeds. Stripping a reservoir leaves its banks more vulnerable to

* See discussion by Hazen and Whipple, E. N., May 6, 1915, and F. P. Stearns, E. N., Aug. 12, 1915. See page 679.

wave action, as seen at Wachusett, but if the material is gravelly or sandy stable shores are soon established naturally.

RESERVOIR CURRENTS

Settling Basin, Kansas City, Mo. Reservoir currents were measured in a new settling basin at Kansas City, Mo., by floats with vanes submerged 4 ft. and 8 ft. This is a rectangular basin, 660 ft. by 450 ft., 24 ft. deep at the toe of brick-paved slopes (1 on 3 and 1 on 2), and 29 ft. maximum depth, with a capacity of 40,000,000 gals. Wash wall, 6 ft. high, is at the top of the slope; 48-in. influent pipe is at the opposite end from the effluent system, which consists of a 36-in. pipe parallel to the short side of the basin, from which a 48-in. effluent pipe leads by means of 48-in. by 36-in. cross at middle of side of basin. Thirty-six-inch elbow at either end of the 36-in. pipe and a 48-in. 45° bend at the cross (throttled to 30-in.) provides three entrances for water. Started at the inlet, an 8-ft. float traversed the outer periphery in 218 min.; a 4-ft. float traversed only the short side adjacent to the inlet, being caught in an eddy in the opposite corner. Floats started at mid-basin circled around the starting point with a tendency to move toward the perimeter. There are no baffle walls; tests showed little use for them. Other tests confirm the complete swing of currents. (E. N., Feb. 29, 1912.)

Erratic Course. If the effluent is at the farthest removed point from the influent, the best possible sedimentation, theoretically, will be obtained. Too little is known, however, of circulation in reservoirs, and it cannot be assumed that water moves with any stated uniformity from inlet to outlet. This is especially true of an immense body of water (J. Waldo Smith).

The erratic course of currents of water from inlet to outlet was indicated by float tests at Boonton reservoir, Jersey City, N. J., in 1906. They indicated no law whatsoever, and on different days under similar circumstances, floats, started at the same point, would take diverse courses. The shape of the reservoir, winds, relation of capacity to quantity flowing through, convection currents in water, and other things affect the direction and velocity of reservoir currents.

SILTING OF RESERVOIRS

In eastern United States little silting occurs. No provision is made against it in most waterworks. Most municipal reservoirs have considerable area and depth below the lowest draft line, so a little silting is not noticed. Kensico Lake, N. Y., an 1,800,000,000-gal. reservoir completed in 1884, silted about 3 per cent. in 23 yrs. Samples taken from four scattered points, Feb. 27, 1907, indicated that the deposit above the original bottom averaged 6 in., (max. 18 in.) partly of organic growth from the overlying water, but principally soil washed in; analyses showed loss on ignition from 6 to 13 per cent., averaging over 10 per cent. (Report by A. Hazen and G. W. Fuller, Apr. 30, 1907.) Williamsbridge and Jerome Park reservoirs, storing water which has already stood in larger reservoirs, show an accretion of $\frac{1}{8}$ to $\frac{1}{4}$ in. of silt per year. They are concrete lined. (E. N., Dec. 31, 1914.) Beaver Dam Creek, North Carolina, brought down 12,000 cu. yd. of sediment in 1 yr. (by cross-sections

taken) from a drainage area, bare of forest, and but 14 sq. mi. in extent. (E. N., Aug. 20, 1908, p. 211.)

Western United States. Filling of reservoirs by silt is a serious question in some parts of U. S., and in other regions having easily eroded soils. La Grange reservoir, Tuolumne river, California, lost 50 per cent. capacity in 10 yrs.; deposition represents 0.000067 volume of stream. Analyses of the water show that silt equivalent to 0.000043 volume is carried by the site. Total silt load is 0.00011 volume of the stream. (E. N., Dec. 10, 1908.) Observed rate of silting in Sweetwater reservoir, California, has been 0.5 per cent. per yr. for the past 20 yrs. (E. R., July 20, 1912, p. 79.) Austin reservoir, Colorado river, with a capacity of 53,490 acre-ft., at the end of 4 yrs., showed a silt accumulation of 22,000 acre-ft., equivalent to 40 per cent. loss.

Silting Laws.* Observations at thirty stations on 90 mi. of silty canals in India (sections and velocities varying) showed a mean "critical velocity" which neither deposits silt already in suspension, nor produces further scour. This critical velocity, V_c depends only on depth of stream, D. $V_c = CD^m$, the coefficient C and exponent m varying with the regimen of the streams; m is approximately 0.64 for all kinds of channels. C depends on materials: light, sandy soil, 0.82; sandy loam, 0.99; material of fineness intermediate between these, 0.90; coarse silt, 1.07.

$$B_c - B = B^c\left[1 - \left(\frac{V}{V_c}\right)^{\frac{5}{2}}\right]$$

B = volume of silt carried at any mean velocity, V.
B_c = volume of silt carried at mean critical velocity, V_c.

$(B_c - B)$ represents the volume of deposition, if positive; of scour, if minus. This formula is useful for determination of silting in laterals, after experiments have given V_c and B_c for the main canal. (E. N., June 8, 1911, p. 701.)

Transportation of Sediment* takes place by dragging along the bottom, or by carrying in suspension. The weight of particles which will be moved by flowing water is supposed to vary as the sixth power of the velocity.

Table 38. Limiting Bottom Velocity for Stability of Material; Observed by Various Engineers

Material	Velocity, ft. per sec.	Material	Velocity, ft. per sec.
Soft earth	0 25	Sea pebbles (1.06 in. diam.)	2.20
Soft clay.	0 50	Brickbats (4.76 cu in)	2 25 to 2.50
Sand..	1 00	Slate (9.06 cu. in)	2 75 to 3.00
Gravels	2 00	Broken stone	4 00

Table 39. Descent of Fine Materials Through Still Water

Material	Velocity, ft. per sec.
Brick clay, mixed with water and allowed to settle half an hour	0 009
Fresh-water sand	0 166
Sea-sand...	0 196
Rounded pebbles (size of peas).	1.000

* R G Kennedy. Proc Inst C. E, Vol. 119, 1895.

Table 40. Velocities Required to Move Solid Particles in Water

Kind of material	Approximate velocity required on bottom: Ft. per sec.	Mi. per hr.
Fine clay and silt	0.25	$\frac{1}{6}$
Fine sand	0.50	$\frac{1}{3}$
Pebbles, $\frac{1}{2}$ in. in diam.	1.00	$\frac{2}{3}$
Pebbles, 1 in. in diam.	2.00	$1\frac{1}{3}$

Removal of Silt by Sluicing. The oldest means for cleaning silt from reservoirs is a scouring gallery, known as a Spanish gate. It consists of an undersluice with an opening at the upstream end from 3 to 10 ft. high, and from 3 to 10 ft. wide. This opening is situated at the bottom, on line with the bed of the original stream. Its size gradually increases downstream; it is closed by a wooden door, supported by horizontal timbers set in apertures. Above the undersluice is a gallery, about 3 ft. wide and 7 ft. high, closed at the upper end and open at the lower to allow workmen to enter. It is connected with the undersluice by an opening 2 ft. in diam. near the door. When the reservoir is empty, the wooden door is put in position from the lower side, supporting timbers are placed, and the whole door well calked. When it is desired to rid the reservoir of silt, men enter through the gallery and cut the supporting timbers. The solidified mud generally holds the door in place and so a long iron pole is used from the top of the dam to loosen the silt. Then the door is swept out together with the accumulations. Whatever silt is left, is shoveled into the stream. This scheme was used in the Mercedes dam, Mexico, with the following modifications: The outlet consists of a stone culvert with semicircular arched roof, about 8 ft. high and 6 ft. wide. Over this outlet rises a rectangular tower, built against the upstream face of the dam, and extending above it. At the base of the tower on the outside is a 6-ft. circular sluice gate, operated from the tower; inside is a similar gate, 5 ft. in diam. By filling the tower through small openings, pressure on the outside gate is relieved and it can be raised. The inside gate, which, being accessible, can be kept well oiled, may now be raised; the silt is carried out with the water. This method wastes great quantities of water, and does not effectively remove all silt. (W. D'Rohan, Eng'g-Contr., Jan. 11, 1911.)

Limits of Sluicing. The removal of silt by sluicing is impossible, except for the limited area adjacent to the sluice gates. In large reservoirs, the greater portion is dropped beyond the influence of sluice gates, nor would much silt be disturbed, were the whole dam removed by a wash-out. When the Austin, Texas, dam failed the reservoir was so silted that its capacity was but 60 per cent. of the original. Very little sediment was removed under these unusually favorable conditions. This might be explained by the fact that the eroding power of water varies as the square of the velocity, and the transporting capacity as the sixth power of the velocity. The fall in velocity when a stream empties into a reservoir causes a deposition, which will require a much higher velocity to pick up again. (J. L. Campbell, Eng'g-Contr., Jan. 25, 1911.)

Prevention of Depositions. J. D. Schuyler recommended for the Mercedes dam a series of small dams above the main dam, to partially precipitate silt. Bhatgarh dam, India, besides two overflow waste-weirs, has fifteen under-

sluices near its center, 12 ft. above the river-bed, to provide passage for early floods at low levels rather than over the high weirs, as this latter would cause most of the silt to be dropped in the reservoir. Sluices carry away the silt at a time of year when most abundant. They prolong the life of reservoirs. This principle was applied to Assuan dam. R. B. Buckley cites following methods of preventing depositions of silt in canals: (1) Works in river to clear the water before it enters the canal. (2) Construct headworks so that only water bearing an allowable proportion of silt is admitted. (3) Construct settling-basin near head of, and in the canal, which can be cleared; or use two supply canals which can be cleaned alternately. (4) Construct a double row of sluices, with settling basins between, water being drawn off through the lower row, and so designed that depositions in the basin can be flushed back into the river. (W. D'Rohan, Eng'g Contr., Jan. 11, 1911.)

FORESTRY AND GRASSING

Reforestation of Reservoir Margins. Tracts around large reservoirs, if left to Nature, are of no economic value, and, if not properly controlled, will become detrimental to the quality of water. Such tracts cannot be put under cultivation or used for grazing because the manurial conditions would be objectionable. The forestation of marginal lands protects the water, prevents wash-outs, and supplies marketable timber. Forests guard against floods, winds, snow slides and erosion, and have great influence over the temperature of the surrounding country. Leaves contain 50 to 75 per cent. moisture. More heat is required to raise the temperature of a pound of water than almost any other substance; hence, leaves absorb more heat than bare rocks or soil. Leaves also tend to keep the air cool by transpiration. Forests have high influence on evaporation. Dr. Ebermayer, German forest meteorologist, showed that evaporation from forest soil without a layer of mold (humus) was 47 per cent. of that from soil in the open, while with a layer of good mold, it was 22 per cent. When rain falls upon a dense forest, from less than 0.1 to 0.25 of it is caught by the trees, the greater part of this being evaporated therefrom.

Economics of Forestry. Pine trees 50 yrs. old will be at least 14 in. in diam. and 60 ft. high. It is profitable to cut trees at 40 yrs., but the value of the increment makes it worth while to wait. At 50 yrs., white pine will have a volume of 46,000 board ft. per acre with a stumpage value of \$10 per 1000 ft., making the value per acre, \$460. A planted forest needs to be thinned at 15 and 30 yrs. An ideal growth for reservoir margins is coniferous, shedding no large leaves; arbor vitæ or white cedars are good. Spacing of trees is 6 ft. apart each way. Deciduous trees should be planted 50 ft. back of the conifers if desired for landscape effects. Seedlings are planted in seed beds, about three seeds to the sq. in. At the end of 1 yr., transplanting takes place. Seedlings remain 2 yrs. in the transplant bed; then they are ready to set out. 30,000 seeds planted in a nursery should produce 10,000 trees. At Wachusett reservoir, the average cost of planting 3-yr. old white pine was \$5.25 per 1000 trees. Trees best adapted for forestation in northern and eastern U. S. are white, red and Scotch pine, European larch, Norway spruce, red oak and sugar

maple. White pine grows in almost any kind of soil. It is desirable for forestation on account of its ease of production and value.

See also, "Reforestation of Watersheds for Domestic Supplies," by F. W. Rane, Mass. State Forester; "Practical Forestry for Water Works," by E. S. Bryant. Jl. N. E. W. W. Assn., 1911.

Field Work. When buying trees and seeds, American species should always be given preference, as foreign stock has brought diseases and insects of most damaging kinds. The long duration of travel is hard on the stock. The most durable kinds of wild stock to transplant are white and red pine, spruce, hemlock, balsam, oaks, hickory, maple and basswood. Transplanting of conifers should begin as soon as frost is out of the ground, and may continue until the buds begin to swell. Conifers should not be more than 2 yrs. old for economical handling, as they have long straggling roots. Seed beds should be enriched by good fertilizer, well rotted barn manure being the best. Compost heaps of manure and earth should be made each year, which will rot, without leaching, 3 yrs. before using. The addition of unleached hardwood ashes is always valuable.

Best trees for ornamental planting are: For avenues, Norway maple, pin oak, English linden, oriental plane, elm and sugar maple. For screens, white pine, Norway pine, Scotch pine, Austrian pine, hemlock, white spruce, Norway spruce, Douglas spruce, North Carolina poplar, Lombardy poplar, willow and white birch. For settings for buildings, red cedar, hemlock, Lombardy poplar, Norway spruce, white spruce, larch, beech, oak, tulip and maple. For park purposes, various shade trees should be used and evergreens named above; for shrubs, preference should be given to those having berries, colored barks, and fine foliage rather than floral display, although beautiful flowers when combined with other good qualities are desirable.

Grassing Cuts. For grassing cuts and fills along highways, etc., use sand clover as well as white clover in sandy places. Honeysuckle, ivy, and some other vines for slopes, require less care than grass and permit less growth of weeds.

Grassing Embankments. For binding embankments, where firm turf is required, Couch or Witch grass is adapted to Northern and Middle States; there are a number of forms, all good hay grasses; objectionable in fallow lands owing to rootstocks. Couch grass makes very tough sod, especially valuable in binding canal banks. Hungarian Brome forms a good turf, but the creeping rhizomes (rootstocks) are not as strong as in couch grass. In southern states, Johnson or finer Bermuda grasses succeed better than Couch grass. Johnson grass produces a mass of widely creeping strong rhizomes, which are hard to eradicate after they have once filled the ground. Bermuda grass does not penetrate as deeply as Johnson; in sandy soils it succeeds better; is of great value as binder on sandy levees; and one of best pasture grasses of the South. If land is moist and soil clayey, knot grass may be substituted for Bermuda; not injured by moderate submersion; is useful as covering for silty deposits on banks. Best method of starting a growth is transplanting rootstocks. Following specification is from John H. Gregory: Surface shall be carefully prepared and raked over, then use at least 50 lb. grass seed per acre, mixture

consisting of 4 parts beach grass, 4 parts Hungarian brome, 4 parts orchard grass and 1 part white clover, together with not less than 400 lb. of raw-bone fertilizer per acre; roll well. Embankments may often be protected by sodding a portion of the area. A levee specification reads: Entire surface shall be planted with living sods of Bermuda grass not less than 4 in. square, placed not more than 2 ft. apart. Sods shall be covered with 1 or 2 in. of earth, as directed. (E. N., July 11, 1912, p. 74.)

Catskill Water Works. On portions of Catskill aqueduct embankments and slopes of cuts, and on earth dikes, the following grass seed mixtures were used:

For tunnel portal cuts and adjacent embankments: 25 lb. red top; 5 lb. Rhode Island bent, 5 lb. Kentucky blue, and 17 lb. of timothy per acre.

For rock débris embankments, containing much fine material, and for borrow pits or spoil banks, 20 lb. per acre, 10 lb. each of trifolium incarnatum (Crimson clover), and Bokhara clover (sweet clover).

For top soil, 69 lb. per acre: 12 lb. agrostis stolonifera (creeping bent); 9 lb. agrostis vulgaris (red top); 3 lb. bromus pratensis (meadow brome grass); 10 lb. festula ovina (sheep's fescue); 15 lb. lolium perenne (perennial rye grass); 5 lb. trifolium repens (white clover); 10 lb. avena elatior (tall rye oat grass); 5 lb. trifolium incarnatum (crimson clover).

For clayey embankments, use 65 lb. per acre: 8 lb. agrostis stolonifera (creeping bent); 8 lb. agrostis vulgaris (red top); 4 lb. anthyllis vulneraria (kidney vetch sand clover); 11 lb. avena elatior (tall meadow oat grass); 8 lb. fetucca durinsula (hardy fescue); 11 lb. poa compressa (Canadian blue grass); 8 lb. poa pratensis (Kentucky blue grass); 2 lb. trifolium repens (white clover); 5 lb. trifolium incarnatum (crimson clover).

For sandy dry banks 60 lb. per acre: 10 lb. creeping bent; 4 lb. kidney vetch; 10 lb. tall meadow oat grass; 10 lb. smooth brome grass; 13 lb. sheep's fescue; 13 lb. creeping fescue.

To the above, 10 lb. of oats were added for spring seeding; 10 lb. of rye for fall. The seed expert at Vaughan's Seed Store, New York City, recommended (summer of 1912) for ordinary sowing, at 35 lb. per acre, the following mixture:

25 lb. Italian rye grass	at $0.085	$2.13
12 lb. timothy (phleum pratense) .	0.20	2.40
45 lb. red top (unhulled) ..	0.12	5.40
15 lb. Canada blue (poa compressa)	0.19	2.85
10 lb. creeping bent... .	0.25	2.50
3 lb. white clover.... . .	0.40	1.20
110 lb. total..........		$16.48

All seed must be fresh and good. Crops of some years of a given variety are not so good as others. Test seeds by forced germination of samples in moist sand. Usually it is prudent to select varieties of seeds commonly sold; they are more likely to be fresh. It is much more difficult to start a good growth on an embankment with the usual steep slopes, because the fresh surface is eroded by rains, and moisture rapidly drains away; to counteract this, tops of Catskill aqueduct embankments were dished slightly in some places; but if the top edges of the embankments were made so high that the top did not become level

and the character of the earth were such as to retain water, the standing water injured the grass.

Examination of the grass growing on embankments led to the suggestion of a simpler mixture, containing two-thirds red top seed, and one-third perennial rye grass, with a small addition of white clover. Red top of good vitality should give satisfactory germination in soils ranging from moist, rich loam to relatively dry clay. For places without top soil, red and Bokhara clover may succeed; the function of the red clover (which winter-kills) is to grow up and form a root substance, which may serve as food for the second season's growth of Bokhara. Sow in late March or early April, at the rate of 10 to 15 lb. per acre.

Prof. E. G. Montgomery, N. Y. State College of Agriculture, Cornell University, recommends: (*a*) for top soil of fair quality, 4 or 5 in. thick: red top (hulled) 8 lb. per acre; Canada blue grass, 15 lb. per acre; Kentucky blue, 15 lb. (*b*) For fairly good top soil, but liable to wash, add 10 lb. of annual Italian rye grass, which persists 2 yrs., forming a sod. (*c*) For very thin top soil, sow Quack grass, but only as a last resort, as it has no value for hay, and is a menace to neighboring hay crops. Alfalfa is not likely to prove of value as it is expensive to seed, does not form good sod quickly, and dies out unless cut regularly.

Volumes of Prismoidal Tanks. For tanks having uniformly sloping sides, and level bottoms, McLean has worked out the following method of finding partial volumes. Divide into equal numbers of zones, each of depth, H. Find volumes of lowest two zones by usual computation, and compute difference. Second difference is constant; for frustrum of pyramid $= 8\ s_s s_e H^3$; for frustrum of cone $= 6.283\ s_c{}^2 H^3$. s_s, s_e and s_c = ratio side slopes, end slope and conical slopes, respectively. To first differences add second differences successively, getting first differences for each zone, and, by successive adding, the volume of each zone. Example: Rectangular reservoir, with all slopes 1 on 2. Bottom dimensions, 286 × 135, depth 14 ft.; find contents of zones 6 in. deep. Second difference $= 8 \times 2 \times 2 \times \frac{1}{8} = 4$. Computation of lowest two zones gives volumes of 19,516, and 19,941, with first difference of 425. Then successive first differences are 425, 429, 433, 437, etc. Tabulation would be as follows·

Zone	Second difference	First difference	Volumes
0 –0 5	4 0	. . ....	19,516
0 5–1 0	4 0	425	19,941
1 0–1 5	4 0	429	20,370
1.5–2.0	4 0	433	20,803

E. N., Jan. 25, 1912

Reserve Storage. In operating the Croton reservoirs of New York City's water system the quantity of water in storage is not allowed to become less than a quantity which, added to the lowest run-off recorded for a year, would equal the anticipated consumption for such year. (W. W. Brush, J. N. E. W. W. A., 1913, p. 424.)

CHAPTER VII

MASONRY DAMS

(For general list of symbols, see page 128)

General Considerations. Here can be given only working formulas with such explanation as may be needed for intelligent use by an engineer who has some knowledge of the subject. In assigning values to permissible pressure on masonry and to ice thrust, and in making various allowances not subject to exact calculation, but which affect the thickness and hight of the dam, the judgment of the engineer must be guided by the importance of the structure, quality of available materials and workmanship, local climate, nature of the site, consequences of partial or total failure, and funds at his disposal. Choice between curved and straight plan depends chiefly upon the character of the site; a narrow gorge with abrupt rocky sides naturally suggests an arched dam; but on many sites an arch-shaped dam not only would possess no structural advantages, but would cost more and in some cases might be less secure because not fitting the foundations so well. Good foundation is one of the most important considerations for a masonry dam, both as to stability and cost. Only relatively low and unimportant dams may be, under stress of circumstances, placed on other than rock foundation, and such other foundation must be most intelligently selected and prepared. Some "hollow" types of dams are useful for such situations, but must be used with discretion.

The first requisite is ample strength; second, materials should be arranged to furnish this strength with minimum cost. Requisite strength, or margin of safety, and permissible cost will vary widely, from the principal dam of a great reservoir for a metropolitan water supply, situated upstream from populous and expensive territory, to an unimportant power dam in the wilderness. It must be borne in mind that worst conditions—conditions which are possible and for which a designer should provide—may not occur for a great many years. That a structure has stood a number of years does not necessarily prove that it is permanently secure.

For conservative practice: (1) Explore very thoroughly all subsurface conditions by test pits and borings, under guidance of a geologist.* (2) Provide for upward hydrostatic pressure, ignoring the possibility that such pressure may be avoided or reduced by cut-off, grouting of seams or other means (applies especially to a site where the rock has horizontal seams). (3) Assume that a few feet of thickness of the downstream face of the dam will be ineffective, in some climates, to resist compressive stresses in cold weather. (4) Make due allowance for ice pressures in cold climates. (5) In computations make conservative assumption as to unit weight of the masonry. (6) Provide for vacuum under the overfalling water sheet in overflow dams. (7)

* Consult "The Principles of Engineering Geology," by Herbert Lapworth, Proc. Inst. C. E., Vol 173, 1908, p 298.

Carry foundations deep enough to insure stability. (8) If the dam is critically located in respect to a community, allow an ample factor of safety.

Wind Pressure on Dams. Wind pressure on the upstream face, reservoir empty, is very much less than that of water, reservoir full, and the pressure of the wind on the portion above full reservoir level is much less than the amount necessary to allow for ice action, while if a strong wind arose with ice covering the reservoir, the wind would probably prevent the temperature of the ice from rising to 32°, thus preventing maximum ice pressure. Friction of wind on the ice would increase the thrust slightly.

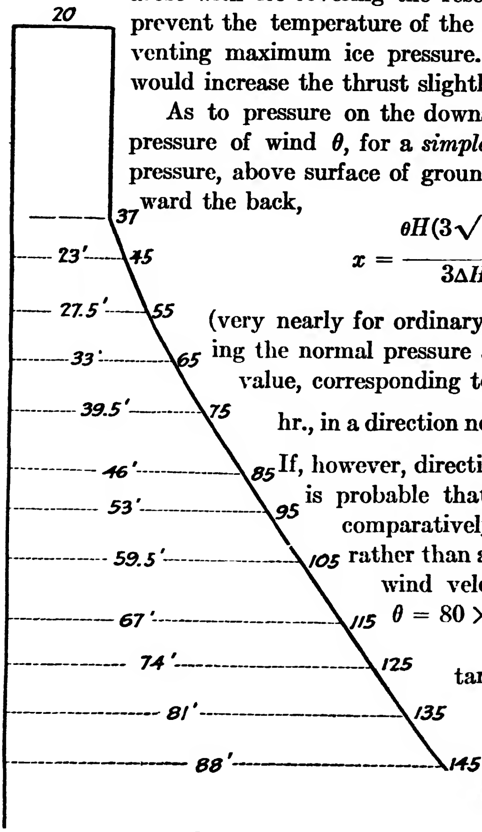


FIG. 37.

As to pressure on the downstream face, calling normal unit pressure of wind θ, for a *simple triangular profile*, the resultant pressure, above surface of ground, reservoir empty, will shift toward the back,

$$x = \frac{\theta H\left(3\sqrt{\rho} - \sqrt{\frac{1}{\rho}}\right)}{3\Delta H + 6\theta} = \frac{4\theta H}{470H + 6\theta}$$

(very nearly for ordinary weights of masonry). Assuming the normal pressure at 80 lb. per sq. ft. (an extreme value, corresponding to a wind velocity of 100 mi. per hr., in a direction normal to the face), $x = \frac{32H}{47H+48}$. If, however, direction of wind is horizontal (and it is probable that in case of dam situated in a comparatively narrow gorge it has an upward rather than a downward tendency) with same wind velocity, normal pressure would be $\theta = 80 \times \sin^3 \phi$.

$$\tan \phi = \frac{1}{\sqrt{\rho}} = \frac{1}{\sqrt{2\ 5}},\ \phi = 32° \ 20'$$

and value of x would become $\frac{52H}{470H+80}$ (Symbols on page 128.) In Fig. 37 or any similar profile, shifting of resultant pressure due to wind will not cause it to leave the middle third for the portion having the top width B until hight h is $> \frac{B^2\Delta}{3w} = \frac{52B^2}{w}$ or for Fig. 37 until H is 260 ft. Below this hight, x for such a profile will lie between the two values found above for the triangular profile, being probably nearer the second values than the first. Therefore, all wind pressures may be neglected.

Upward Water Pressure. Eight cases may be assumed: (*a*) No upward pressure, *i.e.*, masonry and underlying rock impermeable. (*b*) Upward pressure = h at heel, zero at toe, and varying uniformly between, uniformly distributed, and exerted upon one-third area of joint. (*c*) Same as (*b*) but exerted upon two-thirds area of joint. (*d*) Same as (*b*) but exerted upon *full area of joint.* (*e*) Uniform upward pressure = h exerted upon full area

Table 41. Effect of Upward Water Pressure, as shown by Change in Position of Resultant Reaction, and Variation in its Intensity at Toe of Joint

Case			*a*		*b*		*c*		*d*	
Dist. from top to joint	*l*	*l*/3	*u*	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50′	36.0	12.0	12.2	5.3	*9.9	*5.8	*8.7	*6.0	*7 9	*6 2
110′	78.4	26.1	33.3	7.4	30.9	7.4	27.9	*7.4	*22.3	*7 7
170′	132.5	44.2	56.1	11.8	.....	...	....	...	.	
200′	149.7	49.9	59 1	16.4	.			.		
Same, assuming joint does not open with tension at heel when *u*<*l*/3										
			u	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50′	36 0	12 0	12 2	5.3	*11 3	*5 4	*10 2	*5 6	*9 0	*5 9
110′	78.4	26.1	33.3	7.4	30.9	7.4	27 9	7 4	*24 1	*7 6
170′	132.5	44.2	56 1	11 8	.	.			.	
200′	149.7	49 9	59 1	16 4	.					

Table 41. Effect of Upward Water Pressure.—(*Continued*)

Case			*f*		*g*		*h*		*e*	
Dist from top to joint	*l*	*l*/3	*u*	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50′	36 0	12 0					.		7 9	+1 1 5 2
110′	78 4	26 1	.	.	.	.	.		22 3	+3 0 5 5
170′	132 5	44 2	51 7	11 6	46 6	11 3	*35 7	*11 7	35 7	+4 8 8 7
200′	149 7	49 9	55 1	15 7	*47 3	*15 4	*35 7	*16 2	35 7	+5 8 13 0
Same, assuming joint does not open with tension at heel when *u*<*l*/3										
			u	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50′	36 0	12 0			.				7 9	6 3
110′	78 4	26 1		.	.				22 3	8 5
170′	132.5	44.2	51.7	+0 4 11 6	46 6	+0 7 11 3	*39 1	+1 1 11 4	35 7	13 5
200′	149 7	49 9	55.1	+0 7 15 7	*49 1	+1 4 15 1	*40 2	+2 0 15 6	35 7	18 8

* Only quantities marked * are changed in lower part of table Upper figures in "*r*" column under "*e*," "*f*," "*g*," "*h*," show correction for neglecting proportion of pressure at toe directly counteracted by back-water pressure. A *trial* profile for Wachusett dam (see p 190) was used for above computation. Ice thrust (horizontal) = 47,000 lb. Back-water level, from which *h′* is measured, 138 ft. below top. Water level and ice 15 ft. below top. List of symbols, p 128.

of joint. (*f*) Upward pressure = h at heel, h' at toe (backwater, or assumed ground-water level is h' above joint) varies uniformly between, is uniformly distributed, and exerted upon one-third area of joint. (*g*) Same as (*f*) but exerted upon two-thirds area of joint. (*h*) Same as (*f*) but exerted upon full area of joint. Computations in Table 41, shown in columns, *a* to *h*, correspond to cases *a* to *h* respectively. When u is less than $l \div 3$, upstream part of the joint for length $= l - 3u$, being in tension, according to the trapezoidal law, is supposed to be open and so to be subjected to full reservoir pressure, for the upper half of Table 41, but, in the lower half, the joint is supposed not to open under tension. Latter assumption is made to avoid complication introduced by considering $l - 3u$ to be under full head, so that the effect of a given upward water pressure exerted upon increasing proportions of area of joint may be more clearly seen.

Earth Filling on Toe. An advantage of an earth fill over the toe is the diminution of the effects of temperature. As the lower part of a dam carries the higher unit compressive stresses, the earth covering may be a distinct safety factor in high dams. Some engineers have advocated covering a large portion of the downstream face of a dam in this way, where conditions are suitable. Incidentally, the covered portions of the masonry, of whatever kind, need no finish and so a small saving can be effected.

ICE THRUST ON DAMS

Force Exerted. Ice is often injurious to dams, especially to dams of inflexible materials. Several reinforced-concrete dams of the buttress type were badly damaged in winter of 1908 by ice expansion. While in rare instances the river may be so narrow that the shores take the thrust, usually the dam creates a large reservoir and makes an ideal ice field. Various authorities place the shoving force of a field of ice at 12 to 25 tons per sq. ft. Modulus of elasticity of ice is about 180,000 to 360,000 lb. per sq. in., and maximum expansion due to temperature 0.00025 to 0.00066 ft. per ft.

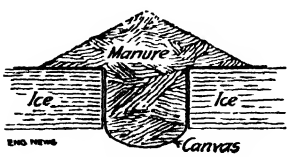


FIG. 38.

At Waldron, Ill., a new dam was built, about 50 ft. below an old one, the old dam not being disturbed. Much trouble had been experienced on the old dam from ice thrust, but after building the new dam, the water passing over old set up such a current that no ice could form between them. This indicates a way to prevent ice from injuring a dam. Another is to keep a channel a few feet wide cut clear across the river and just above the dam. This channel may be filled with manure and covered with canvas to prevent freezing (see Fig. 38). This method was adopted at Mazeppa, Minn., where 36-in. ice is common. It worked perfectly. (Obviously this method is not suited to a waterworks reservoir.) (R. C. Beardsley, in E. N., Apr. 23, 1908.)

At Minneapolis, the minimum temperature Feb. 4 to 15 inclusive, 1899, during injury to dam, was: Feb. 4, −14° F.; 5, − 15°; 6, − 14°; 7, − 17°; 8, − 29.5°; 9, − 33.5°; 10, − 22.5°; 11, − 31°; 12, − 20°; 13, − 8°;

14, +12°; 15, + 17°. The damage was noted on the 15th. Joints were not continuous from back to front at the plane of fracture. This dam was about 18 ft. high at the break and the lower 4 or 5 ft. were not displaced. The dam failed by being weakened along plane *ac* and by combination of sliding and revolving about point *c*. The unmoved stepped part extended 200 ft., entire length of opening. Lowest course rested on ledge; dowel pins freely used; laid in Portland cement. Firm, hard, clear ice, little snow on surface, free from cracks. The resident engineer thought failure was not due to expansion of the ice, but rather to the depressed surface becoming level, as the water rose in the pond on Sundays, the action being on the toggle joint principle. Large temperature changes recorded, however, cannot be wholly disregarded. This dam used only as an impounding dam, not an overfall dam; there was always 7 to 8 ft. of water on downstream side. (E. N., May 11, 1899.)

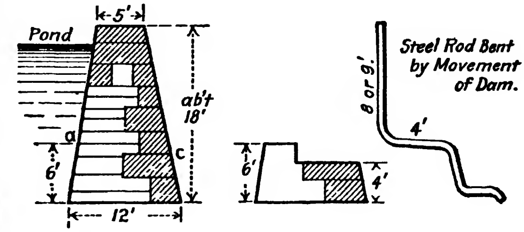


Fig. 39.—Damage by ice to dam at Minneapolis.

At Thomaston, Conn., plenty of evidences were visible of the great pressure exerted by heavy ice during the winter of 1902. At one place where the ice abutted against the end of an immovable wall, while being free to move parallel with the face of the wall, the shearing effect in the ice sheet was obvious. The only bad effect on Wigwam dam, p. 190, was a slight movement of some of the crest stones on the waste weir. These cut stones have a cross-sectional area of about 4 sq. ft., and average 6 ft. long, with a slope of 2 to 1 on the water side. They were set in Portland cement mortar with $1\frac{1}{2}$ in. steel dowels, 3.5 ft. apart built into the wall and extending 5 in. into the stones. It must have taken a large thrust to start these stones on their bed joints after having been set 3 months. (D. W. Cole.) Fig. 40 shows repairs.

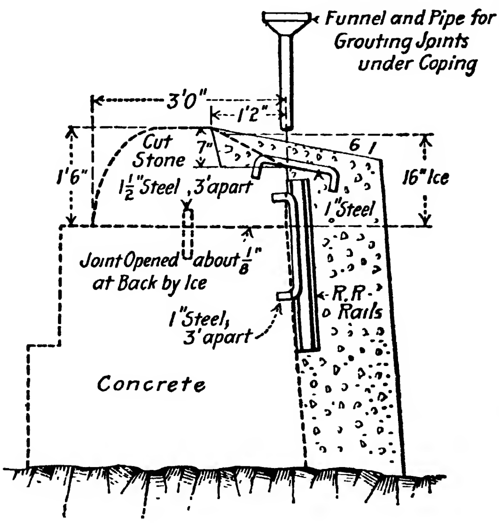


Fig. 40.

Wigwam Dam, Waste Weir, Waterbury, Conn. Original construction in dotted lines; changes and additions in spring 1902, in full lines. Hard ice adhered to back of original coping, as indicated, and shoved and lifted it so as to allow water to squirt in small streams at downstream edge of joint.

TEMPERATURE EFFECTS AND REMEDIES

Experiments: (a) By Thos. U. Walters, architect, made at Girard College, during erection in 1836 and 1837 (Journ. Franklin Inst., Vol. 26; Vol. 22, New Series). (b) By Reuben Shirreffs, at Boston, on a block of concrete. In Girard College experiment a maximum and a minimum thermometer were built into the middle of a brick wall 5 ft. 5 in. thick, exposed on one side to the outer air and on the other to the interior air, there being no artificial heat, and only the protection afforded by a temporary roof. After an entire season, the maximum thermometer registered 61° and the minimum 42°, a variation of 19°. Monthly mean temperatures of air for years 1836 and 1837, according to records of Penn. hospital, Phila., are: minimum, 26° and maximum 74°. Extreme temperatures of any day are given as −3° and 94° F. Shirreffs (1898) applied hot water, of 179° mean, with extreme variations less than 6°, to one end of a concrete block 6 in. square and 30 in. long, for 45 hrs. The other end was exposed to a mean temperature of 68½°, the minimum being 64° and the maximum 73°. The temperature of the block at the beginning was 65°, and it was protected, except at the ends, by 3 in. of carbonate of magnesia. Temperature was observed at frequent intervals at 5 points through holes formed in the block to its middle. From these experiments (ratio of conductivity of rubble to that of brick being taken, from Taschenbuch der Hutte, at from 1.86 to 3.00): (1) A rubble wall 12 ft. thick will be equivalent to a brick wall 5 ft. 5 in. thick, and under the face temperature variation (t), from winter to summer of 48°, variation at middle of the wall (t_m) will be 19°, as in the Girard College experiment. (2) For other seasonal variations, but still the same on both faces, the variation at the middle will be proportional, or for the 12 ft. wall, $t_m = \frac{19}{48} t$, or say $t_m = 0.4t$. (3) For other thicknesses, $2L$, both faces being exposed to same variation, variation at middle will be inversely proportional* to square of thickness, or $t_m = \frac{19}{48} t \left(\frac{12}{2L}\right)^2$ or $t_m = \frac{57}{4} \frac{t}{L^2}$. (4) When variation of external temperatures is not the same on the two exposed faces, the internal change will be minimum at a point such that $L_1^2 : L_0^2 = t_1 : t_0$; L_0 and L_1 being the distances to this point respectively from faces exposed to temperatures t_0 and t_1; it may be calculated by inserting in formula (3) either t_0 along with L_0 or t_1 along with L_1, for t and L respectively, both giving the same t_m. Since $2L = L_0 + L_1$, $L_1 = \frac{2L}{1 + \sqrt{\frac{t_0}{t_1}}}$. (5) The average temperature variation throughout the wall will be: $t_a = 0.78t_m + 0.22t$ and $t_a = 0.78t_m + 0.22\left(\frac{t_0L_0 + t_1L_1}{L_0 + L_1}\right)$, for equal and unequal variations on the two faces. Near the face of portions of the wall exposed to the air, variations will be greater at times and less at other times, owing to constant fluctuations of air temperatures above and below the average. Influence of rapid changes does not penetrate to great depth, and is comprised in the method used. Slight

* (Matter of judgment by Reuben Shirreffs.)

modifications are necessary (*a*) near the top of a dam, because exposed to exterior variation on three sides, and (*b*) near the ground because of the influence of the section above on the section below ground, and *vice versa.*

(See also Bulletin 30, 1913, Iowa Eng. Experiment Station, "The Determination of Internal Temperature Range in Concrete Arch Bridges," by Nichols & McCullough. "Temperature Changes in Mass Concrete," Paul & Mayhew, Trans. A. S. C. E, Vol. 79, 1915.)

Boonton Dam, Temperature Cracks. In the Boonton dam, 2150 ft. long (completed 1903), there appeared, previous to Feb. 27, 1907, 17 cracks of considerable size and same number of smaller cracks; the probable sum of their widths being not far from 3.5 in. Up to March 17, 1906, 5 cracks of measurable size had appeared in *New Croton dam*, 1168 ft. long (finished in 1905); in the spillway section, 1000 ft. long, 10 small cracks. The average distance between main cracks is 100 ft. in the Boonton dam, 200 ft. in New Croton dam. Considerable seepage occurs at Boonton dam, keeping the face wet in summer and ice-covered in winter. Results of thermophone observations in the Boonton dam seem to warrant the following conclusions: In dams of this size, 12 months will elapse between the placing of the masonry and the assumption by the masonry of a temperature dependent on local exterior conditions; in other words, the heat generated by the setting cement only becomes negligible after 1 yr. After the first year, the temperature of the interior will vary 30 days behind the atmospheric temperature. For distances up to 20 ft. from the face, the range of interior temperature varies nearly inversely with the cube root of the distance from the face. The temperature of a mass of masonry due to chemical action is highest 18 hrs. after placing, averaging about 100° F. More cracks appeared in masonry laid in summer months. Approximate computations indicate that the ultimate tensile strength of the mass of a Portland cement dam is not far from 800 lb. per sq. in. (See Fig. 83, p. 182.)

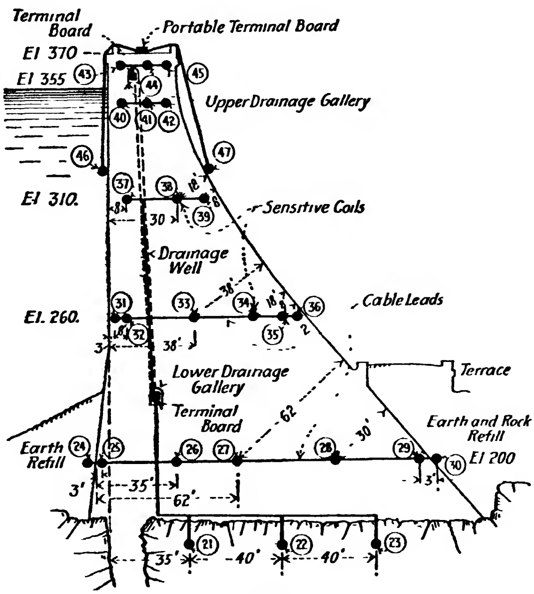


FIG. 41.

Thermophones in Kensico Dam, Fig. 41. Each thermophone consists essentially of two coils of many turns of fine wire of two metals, copper and German silver, having different temperature coefficients of electrical resistance, connected in circuit with batteries and a galvanometer. The wires are

of such size and length that a small change in temperature causes measurable change in electrical resistance, which is indicated by the galvanometer, or detected by a telephone by moving the pointer of a Wheatstone bridge until the silent point is reached. Each coil is enclosed in a brass sheath about $\frac{1}{2}$ in. diam. and 8 in. long. To the sensitive coil is securely soldered a cable of insulated #16 copper wire protected by a lead covering about $\frac{1}{16}$ in. thick. The cable weighs approximately $\frac{1}{2}$ lb. per ft. The equipment consists of sensitive coils embedded in the masonry and connected by cables leading through the masonry to terminal boards at convenient places where indicating instruments may be attached and observations taken. (See also E. N., Dec. 10, 1914.) For description of electric thermometers used in Arrowrock dam, see Trans. Am. Soc. C. E., Vol. 79, 1915, p. 1227.

Temperature Cracks in a Large Western Dam. Slight cracks developed early in some large dams are believed to indicate that rapid progress has a decided influence upon the interior temperature of a mass of cement masonry, and the development of contractile stresses early in the life of the structure, especially when masonry is placed wholly during successive warm seasons and interrupted during intervening winters. Observations of temperature in three lines of pipe embedded in a large western dam, and on top of concrete under protective covering, indicated that maximum temperatures of not less than 100° F. were attained in from 4 to 10 days, maintained as high as 90° F. for about 2 weeks, gradually cooling down to normal in 4 to 6 weeks. High interior temperatures are developed irrespective of atmospheric conditions; hence, concrete while still green and in semiplastic condition is expanded to its greatest possible volume. Then before attaining full strength, it is subject to violent contractile stresses attendant upon its rapid approach to winter temperature.

Longitudinal Cracks. It has been suggested that alternate sections of a masonry dam between transverse expansion joints might be built to the top before intervening ones are started or built to any hight. Since sections first built will cool before the intervening ones are built, intervening sections will be keyed to the older ones, but while setting will be at much higher temperatures, due to chemical action. When they cool, they will attempt to contract, but will be constrained by the vertical tongue and groove, and probably will crack longitudinally (parallel to length of dam), in large dams.

Temperature Reinforcement, Cross River Dam. One and one-fourth-inch square steel bars were placed in horizontal layers in the upper 30 ft. Commencing at the top, these layers included 6 rows of rods in each of the first 2 courses, 5 in the third and fourth, 4 in the fifth, and 2 in the sixth. Nevertheless, temperature cracks appeared, three extending 70 ft. from the top, and a fourth, 43 ft. The cracks were widest at the top; total opening of all cracks was $\frac{7}{8}$ in. Greatest leakage was 23.9 gals. per min. Cracks were calked on the upstream face with lead wool and grouted. Leakage was cut down to 2 gals. per min and was gradually diminishing. No horizontal cracking was indicated. Croton Falls dam was reinforced (see p. 183), but had 3 major and several minor cracks after the second winter following completion.

Temperature and Disintegration Allowance. A convenient method of taking account of temperature effects, possible disintegration due ot freezing of

water in joints or in pores of stone and mortar at the face of the dam, and the character of facing and mode of applying, is to assume that a layer of masonry exposed to the atmosphere cannot be considered as effective for resisting stresses other than those due to these causes. For conditions similar to north-eastern U. S., a thickness of 4 ft., normal to the face, would comprehend the greatest range in temperature, viz., 12° either way from the mean at the inner side of the layer and 32½° either way from the mean at the exposed surface; the masonry back of the 4-ft. line would be subject to much smaller variations, which would produce stresses not excessive in good masonry. Extreme diurnal variations would be much greater than the averages named, but they would penetrate to slight depths. Assuming a variation of 12° from the mean at the 4-ft. line, a coefficient of linear dilatation* of 0.000,004 per deg. F., modulus of elasticity of 360,000 tons per sq. ft., and adjustment to a state of no internal stress at the mean temperature, there would be stresses of 17 tons per sq. ft., either tension or compression. But some experiments show much lower and others much higher moduli, so that the stresses may be estimated as low as 10 or as high as 30 tons for a variation of 12°. Movements caused by these stresses would be very small (for a mean value, about $\frac{1}{250}$ in. in 100 ft. for each ton per sq. ft.); and if the effect be distributed over a number of joints, as seems most probable, the opening, in case of tension, at any one joint would be minute. But since temperature stresses tend in part to relieve those caused by external forces, it is sufficient to assume a line 3 ft. back from, and parallel to, the portion of the face exposed to atmosphere. Although this layer is not included in the section for resisting external forces, its weight is borne by the "effective" masonry, for the face layer is not detached nor self-supporting. So small a part of the upstream face will usually be exposed to the atmosphere, and when so exposed will be subject to no stresses other than those due to the weight of the masonry and to temperature, that it is unnecessary to make a similar allowance here. Having thus provided for temperature, it is reasonable to allow higher unit stresses in the "effective section;" for in methods which make no separate allowance for temperature these stresses, along with indeterminate elements, are provided for by adopting low unit working stresses.

Table 42. Stresses Normal to Direction of Resultant, at Point near Toe of Joint, Making Allowance for Temperature and Frost, by Assuming "Ineffective" Layer, Normal to Face of Dam:

Joint	3-ft. allowance	4-ft. allowance
30 ft. below top	31 4 tons sq. ft.	† .. .. .
50 ft. below top	13 0 tons sq. ft.	20 7 tons sq. ft.
70 ft. below top	10.0 tons sq. ft.	12 2 tons sq. ft.
90 ft. below top	8 0 tons sq. ft.	10 0 tons sq. ft.
110 ft. below top	7.7 tons sq. ft.	9.1 tons sq. ft

Assumed external forces are water and ice at 15 ft below top and 2/3 upward water pressure; cross-section used for computation is that of Wachusett dam. (See p 190)

Maximum pressures at the faces, computed in accordance with the trapezoidal law (p. 127), are probably in excess of the actual, except as greater pressures may be produced by constructing the faces of less compressible

* Coefficient of linear dilatation is expansion per unit along any line, or direction
† Resultant falls beyond limit of joint as reduced, i.e., within 4-ft "ineffective" layer

masonry than the interior, or by the expansion of the masonry in warm weather, or possibly when wet and frozen.

COMMENTS ON UPLIFT, EARTH AND ICE PRESSURES, AND FOUNDATIONS*

High dams on good foundations, in design of which no uplift was directly considered, have stood for years. The amount of uplift should be taken in increasing order for granite, sandstone, stratified rock with horizontal seams, earth and gravel. When saturated earth rests against the dam, it is a common error to find the weight per cu. ft. of earth in water, by subtracting from its weight in air the weight of a cu. ft. of water; if the weight of earth in air is 100 lbs. per cu. ft., the weight in water will be $100 - 62.5 = 37.5$ lbs. Assume that the earth has 40 per cent. voids; then 1 cu. ft. contains 0.6 cu. ft. of solids, and the buoyant force of the water is the weight of an equal volume of water, or $0.6 \times 62.5 = 37.5$ lbs.; hence the true weight of the earth in water is $100 - 37.5 = 62.5$ lbs. In computing thrust due to saturated earth against a dam, this weight must be used if the full head of water is supposed to act on the dam to the bottom of the earth, and this weight should be added to the water pressure. (Wm. Cain.) See Fig. 46 for an illustrative diagram of pressures. This detail should receive consideration if any relatively large depth of earth is to be refilled or embanked against the upstream face of a masonry dam, as is not uncommon, particularly with non-overflow dams. If earth be selected and thoroughly compacted in thin layers, tight against the dam, the pressure may be less than the fluid pressure of saturated earth above indicated. With deep foundation and high ground water, saturated earth pressures will need attention at the downstream toe.

Austin, Pa., dam did not go down into rock, but only rested on or near surface. These rock layers were nearly horizontal, from 2 to 6 in. or more in thickness, and parted by unctuous clay. The coefficient of friction between two surfaces of rock separated by clay or soft, wet shale may not be more than from 0.3 to 0.5, while the dam would probably have failed if the coefficient had been 0.55. (Descriptions in E. N., Oct. 5, 1911; E. R., Oct. 7, 1911.)

If a dam is built of good concrete or other masonry with good Portland cement mortar, and section is ample, there is no danger of defects occurring in structure itself which will cause failure, such as temperature cracks or cracks due to placing new work on work which has been finished for a considerable time. These should be provided against so far as possible, but they have never been known in themselves to cause failure of dam or retaining wall. If a dam without reinforcement were finished in warm weather, it is almost certain that vertical transverse cracks would occur as soon as cold weather comes. These are large enough at times to permit considerable seepage. Before winter is over these cracks close, leakage disappears, and next season they can hardly be found. Filling is probably efflorescence of magnesia or lime from cement; therefore, temperature cracks are not of sufficient consequence to war-

* Paragraphs under this heading are partly from discussions of "Provisions for Uplift and Ice Pressure in Designing Masonry Dams" (C. L. Harrison, author) in Trans. Am. Soc. C. E., Vol. 75, 1912, pp 142–219.

rant expansion joints; reinforcement required to prevent them would be extremely large, and results obtained would not warrant expenditure. If dam is built of concrete and contains very large proportion of bowlders or heavy rocks of good quality, it is possible that cracks will not occur, but this opinion is not based on numerous enough cases to be reliable. No engineer of experience cares to build a masonry dam at all unless he can found it on material which has sufficient bearing power. Limit might be considered shale rock, which would bear safely at least 10 tons per sq. ft. Such rock is not capable of withstanding erosion due to falling water, therefore an independent spillway must be provided. Where this is impossible and it is necessary for water to pass over dam, provide an apron of heavy stone in mortar, 10 ft. or more thick; upper surface of apron should be below water level and carried downstream 20 ft. or more, depending on hight of dam.

To prevent water passing under the dam, a cut-off must be sunk to reasonably impervious material. Such a trench need not be more than 6 ft. wide, and should be under the heel. The main portion of the dam should go deep enough to have a sufficient barrier of natural material to prevent sliding. In most cases 10 ft. in rock will be sufficient, but if the dam is very high a greater depth becomes necessary. Downstream from cut-off, place longitudinal open drains and connect at frequent intervals with transverse drains extending downstream, to eliminate upward pressure. This can be done without material increase in cost, and is better than to assume excessive upward pressure and build the dam of sufficient section to resist it. (J. W. Ledoux.)

Crystal Springs concrete dam, San Mateo, Cal., was built by Spring Valley Water Co., San Francisco, 1888. The foundation when stripped was found to consist of fair quality of argillo—calcareous sandstone—full of seams and fissures, but all well filled with calcareous cement. Sandstone was porous, and, when exposed to water, soon became saturated. The dam was built of same stone crushed to small sizes. The engineer, Hermann F. A. Schussler, gave dam such cross-section as would stand with safety pressure from the reservoir as well as uplift on entire base. Ice never causes trouble at and near San Francisco. This dam had severest test during earthquake of April 18, 1906. The enormous ground fissure or geological fault passed through the reservoir close to the heel of dam, cutting off the tunnel which delivered the main supply to San Francisco. It cracked and shattered bedrock under the dam, and gave rise to numerous new springs in creek bed below; but with lapse of time these choked up and now discharge an insignificant quantity. Dam with 30,000,000,000 gals. of water behind it, stood earthquake shock perfectly. It was built in blocks, from 50 to 100 tons each, molded in place, interlocked and dovetailed in such a way that there would not be ony continuous seam or joint through the dam. In addition, in each block were twisted iron rods with ends sticking out to tie into neighboring blocks. In spite of all care, when the lake was filled, a leak was discovered 90 ft. below the crest, where the concrete was nearly 100 ft. thick. It ultimately became a mere "weep," overgrown with algæ. This dam is on a curve upstream, the arch having a rise of one-eighth of the length, and the center line radius is 632 ft. (L. J. LeConte.) See p. 172 and p. 186.

DESIGN OF GRAVITY DAMS

Failures and Safe Design. A gravity masonry dam may fail by: (a) Overturning about edge of any joint, due to line of action of resultant passing beyond limits of stability; (b) crushing of masonry or foundation; (c) shearing or sliding on foundation or any joint, due to horizontal thrust exceeding shearing and frictional stability; (d) rupture of any joint due to tension. Hence the following limitations are generally established: (1) Lines of pressure, for reservoir full or empty, must not pass outside middle third of any horizontal joint. (2) Maximum normal working pressure on any horizontal joint must never exceed certain prescribed limits, either in masonry or in foundation. (3) Coefficient of friction in any horizontal joint, or between dam and foundation, must be less than tangent of angle which resultant makes with vertical.

Design of High Dams by Zones. De Sazilly (1853) and Delocre (1858) sought equations by which "theoretical profile of equal resistance" could be determined for any dam. This was found impracticable, and recent methods of computation are simpler than those of the early French dam designers. However, Furens, or St. Etienne, dam (see page 183), prototype of most modern high masonry dams, was based on Delocre's studies. The type of cross-section for usual conditions being now well known (see Fig. 42) the simplest procedure is to draw a trial section, test it by the formulas and modify until all conditions are satisfied. For a low dam (not a *very* low one) use upper part of cross-section for a high dam subject to similar conditions. Formulas and discussions which follow apply to the now generally adopted type of retaining, gravity masonry dams. With various modifications, the cross-section is a triangle, essentially, although the top always has substantial width* and one or both faces may be curved or broken lines. With proper interpretation many of the formulas can be applied to dams of different shape. A very high masonry dam, for purposes of computations relating to design, may be divided into five horizontal zones, beginning at top, in each of which the limits of all conditions included in computations remain constant. The level at which one or more of these limits is reached fixes the bottom of each zone. Low dams may have only the upper one or two zones. As will appear below, hights of zones depend upon allowed unit pressures in the masonry and upon other elements of design.

Zone 1.—A low dam or top portion of a high dam; rectangular cross-section (or substantially so); both faces vertical or with only *slight* batter. Limited by level at which downstream face begins to slope in order to keep point of application of resultant of forces on joint within middle third with reservoir full.

Zone 2. Bottom is level at which upstream face begins to slope to keep point of application of resultant of forces on joint within middle third with reservoir empty.

Zone 3. Bottom is level below which downstream face slopes more flatly than in zone 2, limit of pressure intensity on masonry at toe having been reached for this slope with reservoir full.

*See "The Economical Top Width of Non-Overflow Dams," W. P. Creager, Proc. A. S. C. E., Nov., 1915.

Zone 4. Bottom is reached when pressure intensity on masonry at heel attains limit with upstream slope being used: reservoir empty. Below this level upstream face must have flatter slope.

Zone 5. Slopes of both faces fixed by allowed pressure intensities on masonry at toe and heel, respectively; reservoir alternately considered full and empty. Limited only by necessities of dam site or strength of masonry. Only very high dams have fifth zone.

Controlling Factors. For lower part of a high dam, when choosing p and q, and slopes of faces, the bearing capacity of the rock foundation must be taken into account. It is convenient, especially for construction, to make upstream face vertical to as great depth as is permissible. Considering weight of masonry

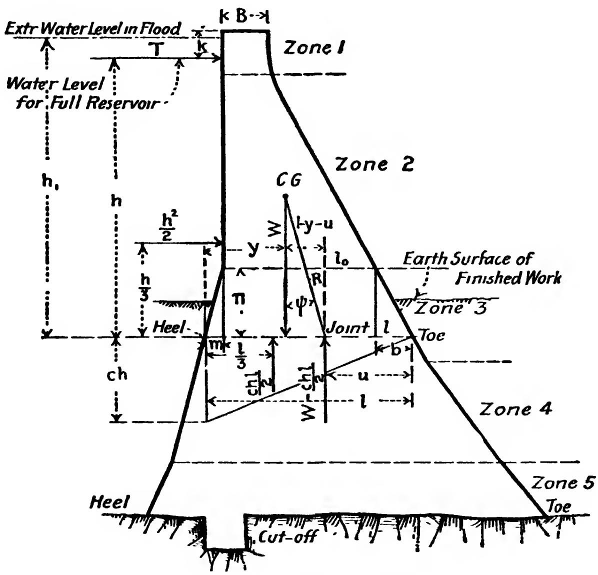


FIG. 42.—Typical section.

only and assuming uniform distribution of pressure over whole base or horizontal joint, the limit of hight with vertical face might be limit of pressure in lb. per sq. ft. ÷ wt. of masonry per cu. ft. Adopting 20 tons and 156.25 lb. respectively, hight = 256 ft. Other considerations greatly reduce this hight. Flood-level conditions control beyond a certain hight (usually greater than 100 ft.) depending upon excess of h_1 over h, and values assigned to T, B and k. In climates where T may be neglected h_1 is used in determining the section, for a dam stable with h_1, would be more stable for h. Having allowed for floods and waves, choice of k is arbitrary or limited by some local condition; B is fixed by necessities of footway or wagonway, usually; but k and B together are very directly affected by T.

Trapezoidal Law of Distribution of Pressure. Assumption: incompressible wall on incompressible base. If W is vertical resultant of pressures, its distribution is represented by trapezoid with p proportional in length to intensity of pressure at one end of base, and q proportional to intensity at other end.

If $y = \frac{l}{2}$, $p = q = \frac{W}{l}$; trapezoid becomes rectangle.

$y > \frac{l}{3}$, $q = \frac{2W}{l}\left(2 - \frac{3y}{l}\right)$; $p = \frac{2W}{l} - q$

$y = \frac{l}{3}$, $q = \frac{2W}{l}$; $p = 0$; trapezoid becomes triangle.

$y < \frac{l}{3}$, $q = \frac{2W}{3y}$; p becomes negative and there is tensive stress represented by the small triangle above the base, its maximum intensity being $-p = \frac{2W}{3y} - q$. Actually the distribution of pressure is probably represented more nearly by the figure with the curved bottom, and ends less than p and q, but we have not the knowledge necessary for constructing this figure.

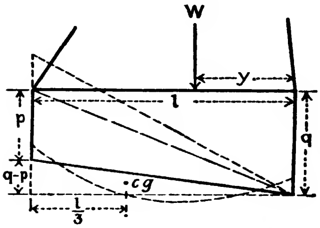


FIG. 43—Trapezoidal law of distribution of pressure.

Formulas for Intensity of Vertical Pressure at Faces. Resolve the resultant of all the forces assumed to act upon the dam in any given case into a vertical and a horizontal component. Consider successively assumed portions of the dam above convenient horizontal joints. Regard the horizontal component as uniformly distributed over the whole joint. Regard the vertical component as distributed in accordance with the trapezoidal law—a straight line distribution. The intensities of the vertical pressure at the face of the joint nearer to the resultant will, on this basis, be given by the following formulas:*

1st Case. $f > \frac{l}{3}$.

$$P_r = \frac{W_o}{16l}\left(2 - \frac{3f}{l}\right)$$

2d Case. $f = \frac{l}{3}$.

$$P_r = \frac{W_o}{16l}$$

3rd Case. $f < \frac{l}{3}$.

$$P_r = \frac{W_o}{48f}$$

Symbols for Masonry Dam Formulas (see Fig. 42) grouped alphabetically. Unit of weights and forces is weight of 1 cu. ft. of water. All lengths are in feet. (These symbols apply only to pp. 115 to 133.)

B = breadth, or thickness, top of dam.

b = horizontal distance between toe of l and toe of l_o (may = 0 for upper part of dam if downstream face is vertical for any distance below top).

c/g = center of gravity.

c = coefficient of upward water pressure.

ch = intensity of pressure of water vertically upward at upstream side of joint; it is considered to decrease uniformly to zero at downstream side.

* It is often convenient to express W in cu. ft. of water, W_o. If W is in same units as P_r, multiply result by 32.

d = depth of water on top of dam.
D = horizontal dynamic thrust of water on back.
E = thrust of earth on downstream face.
f = distance from nearer face of joint, to point of application of resultant.
h = head, or vertical depth, from water level for full reservoir to any joint, real or imaginary, under computation.
h_1 = head from extreme water level in flood, to joint, or to surface of mud.
h_2 = depth of mud against upstream face.
$h^2/2$ = horizontal thrust due to static pressure of water against the upstream face of the dam.
$h_1^2/2$ = ditto for water at flood level.
$H = h + k$ = vertical distance from top of dam to joint under computation.
h' = head due to back-water, down-stream face.
H_1 = depth of fill, downstream side, above joint.
k = hight of dam above full reservoir level.
l = length of joint under computation.
l_o = known length of joint next above, *i.e.*, the one just previously determined.
m = distance from heel to line of vertical upstream face (= zero for portion having vertical upstream face).
M = moment of horizontal force about any point in l.
n = vertical distance between joints l and l_o.
p = intensity of vertical pressure at toe.
P_r = max. intensity of vertical pressure at face of joint, tons per sq. ft.
q = intensity of vertical pressure at heel.
r = intensity of pressure at toe of joint, parallel to R.
R = resultant of forces on joint, reservoir full.
R' = resultant of forces on joint, reservoir empty.
s = intensity of shear on joint, tons per sq. ft.
T = thrust of ice at water surface, considered horizontal (never combined with h_1).
u = distance from toe to intersection of R with joint (when $u < l/2$ it has same value as f).
V = downward water pressure on upstream face.
V_1 = downward water pressure on downstream face.
v = distance between intersections of R and R' with l ($= l - y - u$).
w = weight of masonry above joint; for slice of dam 1 ft. thick, $w = \Delta$ times area of cross-section above joint.
w_0 = weight of masonry above l_o.
W = algebraic sum of all vertical forces acting on joint.
y = distance from heel to intersection of W or R' with joint (when $y < l/2$, it has same value as f).
y_o = value of y for joint l_o.
z = distance from heel to line of action of V.
Toe = downstream edge of joint; Heel = upstream edge of joint.
α = angle of slope of downstream face from horizontal.
γ = unit weight of water = 62.5 lbs. per cu. ft.

γ' = unit weight of mud = 75 to 90 lbs. per cu. ft.
δ = angle of E from horizon.
Δ = unit weight of masonry, taken as 156.25 lbs. per cu. ft. = $2\frac{1}{2}\ \gamma$ in many computations
θ = unit wind pressure (page 116).
ρ = specific gravity of masonry, taken as 2.5.
ϕ = angle of slope of downstream face from vertical.
ψ = angle between R and vertical.

Procedure. Horizontal plane joints are assumed at various convenient elevations, or distances below top, for purposes of computation. Moments of forces taken about intersection of line of action of W with joint, the forces being represented by their resultants and considered to act in the same vertical plane. A cross-sectional slice 1 ft. thick is used for computation and so the forces are those acting upon 1 linear foot of dam. Begin with assumed profiles and modify, if necessary. The following formulas determine the position, under various assumptions, of the intersection of the resultant of the forces acting on the dam with the joint in question. In other words they determine the position of the "line of pressure."

General Equations for Stability against Overturning.*

$$Th + \frac{h^3}{6} + \frac{chly}{2} - \frac{chl^2}{6} - (l - y - u)\left(W - \frac{chl}{2}\right) = 0.$$

1st Case.—Water pressure only, water at full reservoir level. $T = O.\ c = 0.$

$$l - y - u = \frac{h^3}{6W} \text{ or } \frac{h^2}{6\frac{W}{h}}$$

2d Case.—Water pressure and ice pressure, water at full reservoir level. $c = 0.$

$$l - y - u = \frac{h}{6W}(6T + h^2) \text{ or } \frac{6T + h^2}{6\frac{W}{h}}$$

(For any dam, $6T$ is constant, after a value for T has been selected.)

3d. Case.—Water pressure and upward pressure beneath joint. $T = 0.$

$$l - y - u = \frac{h^2 + cl\,(3y - l)}{6\frac{W}{h} - 3cl}$$

4th Case.—Water pressure, ice pressure and upward pressure.

$$l - y - u = \frac{6T + h^2 + cl\,(3y - l)}{6\frac{W}{h} - 3cl}$$

5th Case.—Water pressure only, water at flood level. $T = 0.\quad c = 0.$

$$l - y - u = \frac{h_1^3}{6W} \text{ or } \frac{h_1^2}{6\frac{W}{h_1}}$$

Arrangement of Computations. Tabulate the computations made with the formulas above as indicated on the opposite page. Compute the loca-

* T and W in these equations are in terms of weight of 1 cu. ft. of water.

tion of the center of gravity of the portion of the dam above each assumed joint by moments, to determine y, and check by cutting the section out of cardboard and suspending it.

(Name of Dam). Trial Section No. ——. Stability against Overturning Cases 1 to 4

El. of joint	W	h	h^2	$6\frac{W}{h}$	l	y	cl	$3y-l$	$l-y-u$				u			
									1	2	3	4	1	2	3	4

Case 5

El of joint	W	h_1	h_1^2	$6\frac{W}{h_1}$	l	y	$l-y-u$	u

Another General Formula is:

$$u = l - y - \left\{(h_1 + h_2)[3h_1h_2 + 6T - 3d^2 + cl(3y - l)] + h_1^3 + \frac{h_2^3\gamma'}{\gamma} + 2d^3 + 3D(h_1 + 2h_2 - d) - 6V(y - z) + 6E\left[(l - y)\sin\delta - \frac{H_1}{3}\frac{\sin(\delta + \alpha)}{\sin\alpha}\right]\right\} \div [6(V + E\sin\delta + w) - 3c(h_1 + h_2)l.]$$

If any condition is to be omitted, its factor or term becomes 0 and the general formula must be simplified accordingly. *General Conditions:* (see Fig. 44). Horizontal static water pressure on back (head $= h_1$); upward water pressure on base, pressure intensity decreasing uniformly from $c(h_1 + h_2)\gamma$ at heel to zero at toe; mud (liquid) pressure on back (head h_2); dynamic pressure of water, $D\gamma$; water flowing over top of dam, weight of water, of depth d, on top of dam being neglected; for condition of water not overtopping dam, $d = 0$, and $D = 0$; no dynamic pressure, $D = 0$; no upward water pressure, $c = 0$; no mud (*i.e.*, mud replaced by water), $h_2 = 0$. If ice be disregarded, $T = 0$. If H_1 is of great depth and the downstream slope varies considerably, an approximate solution is possible by assuming an average slope beneath the earth. The expression for earth thrust is general. After u is determined for each joint, p can be determined, the general expression corresponding to above expression for u being:

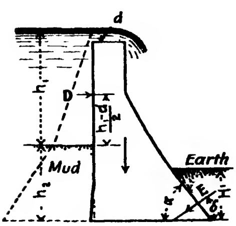


FIG. 44.

$$p = \frac{2\gamma}{l}\left[V + E\sin\delta + W - \frac{c(h_1 + h_2)}{2}l\right]\left(2 - \frac{3u}{l}\right)$$

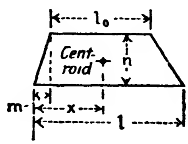


FIG. 45.

In computations for p it is necessary to obtain the position of the centroid of a trapezoid with respect to the heel of the joint. In Fig. 45,

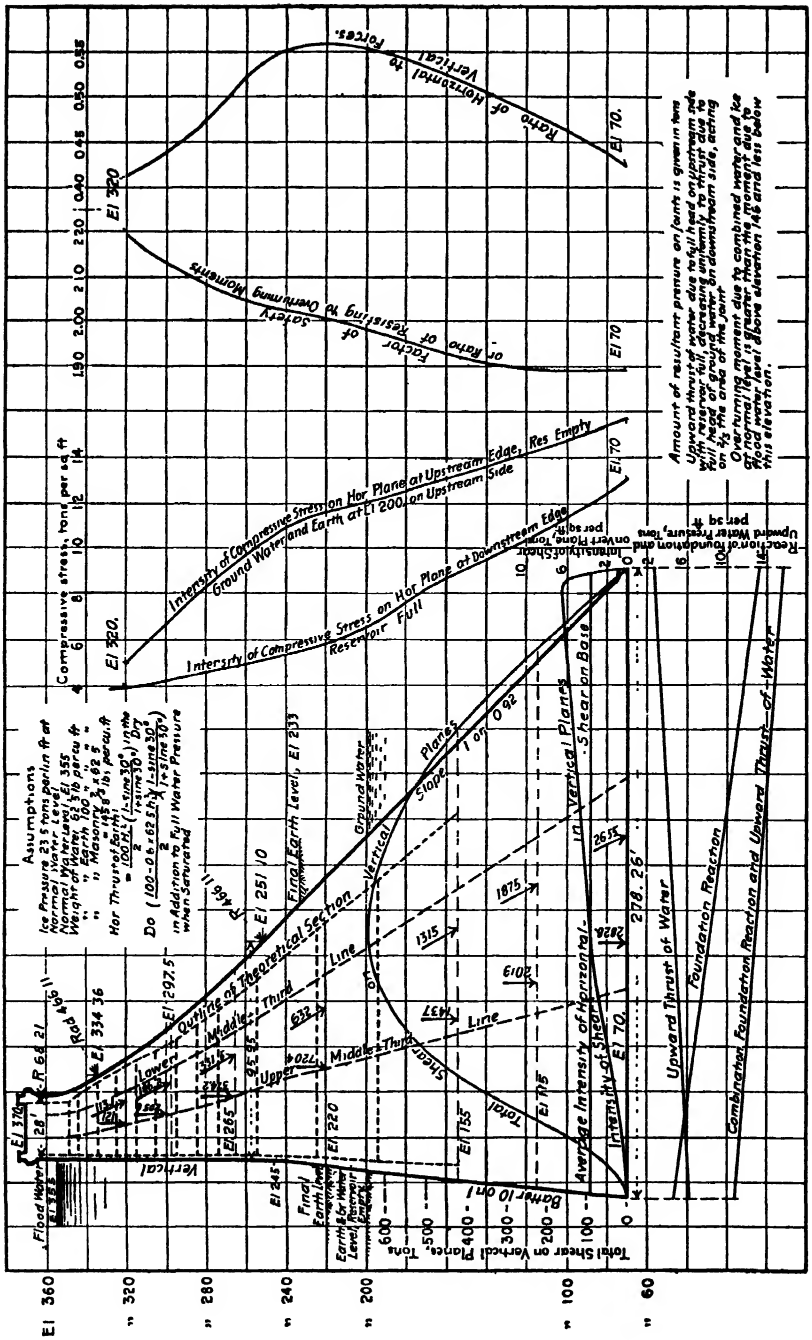


FIG. 46.—Kensico dam at maximum section. Investigation of stability under various conditions.

$$x = \frac{(l^2 + ll_o + l_o^2) + m(l + 2l_o)}{3(l + l_o)}$$

Principal Normal Stresses at Any Point in the Dam and the Planes on Which They Act.* In prism, ABC, Fig. 47, let AB be one of the planes on which stress is normal. Let f be its intensity.

p = vertical unit stress on a horizontal plane;
P_a = horizontal unit stress on a vertical plane;
q = shearing unit stress on horizontal or vertical planes.

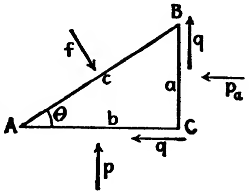


FIG. 47.

Stress on plane, AB, of unit length perpendicular to plane of paper, is thus fc; its vertical component is $fc \cos \theta = fb$, and its horizontal component is $fc \sin \theta = fa$. Therefore

$$\tan 2\theta = \frac{2 \tan \theta}{1 - \tan^2 \theta} = \frac{2q}{p - P_a}.$$

The equation, $f = \frac{1}{2}[p + P_a \pm \sqrt{(p - P_a)^2 + 4q^2}]$, gives the two values of f corresponding to the two planes mentioned; compressive when f is positive, tensile when negative. There can be no tension when $pP_a \geq q^2$.

Usual Methods of Design Criticized. L. W. Atcherley, in paper "On Some Disregarded Points in Stability of Masonry Dams,"† takes exception to current practice in design and supplements generally accepted ideas as to distribution of normal stress on horizontal planes by an assumption as to the shear on these planes

$$cd = \frac{l^2}{12} \quad (1)$$

$$C_{max} = \frac{W}{A}\left(1 + \frac{6c}{l}\right) \quad (2)$$

$$T_{max} = \frac{W}{A}\left(\frac{6c}{l} - 1\right) \quad (3)$$

$$S = \frac{3P}{2A}\left(1 - \frac{4y^2}{l}\right) \quad (4)$$

c = distance along horizontal joint from centroid to point of application of resultant;
d = distance along horizontal joint from centroid to point locating neutral axis;
l = length of horizontal joint;
C_{max} = maximum compressive stress on joint;
T_{max} = maximum tensile stress on joint;
W = vertical component of resultant force acting on joint;
A = area of joint;
S = shear at any point y in joint;

* William Cain, "Stresses in Masonry Dams" T. A. S. C. E., Vol. 64, 1909, p. 208, to which reader is referred for discussion of shears.
† Dept. of Applied Mathematics, Univ. College, Univ. of London. Drapers' Co., Research Memoirs, Technical Series II, London, 1904.

P = total shear on joint;
y = distance from centroid to any point on joint.

Atcherley believes that these equations more nearly express conditions of equilibrium than the usual ones do, even though the latter tacitly assume the first three by imposing the condition of the middle third, and use a friction condition, instead of one for shear as expressed by Equation 4. According to Atcherley there is no reason why dams should be computed solely by testing horizontal joints and asserting that the line of resistance must lie in the middle third, while stresses across vertical joints are neglected. If the problem is to be solved on the assumption that a dam is an "isotropic and homogeneous" structure, general equations for stresses can be determined only by the following considerations: (*a*) Normal and shearing stresses on horizontal top and curved down-stream face are both zero. (*b*) Normal stress on battered up-stream face is equal to water pressure, and shear is zero. (*c*) Either stresses or shifts† be supposed given over the base. It follows at once from this that Equations 1, 2 and 3 are not absolutely true, but that the shear is fairly closely represented by Equation 4.

From his investigations Atcherley concludes: (1) Current theory of stability of dams is both theoretically and experimentally erroneous, because (a) theory shows that vertical and not horizontal sections are critical sections, and (b) experiment shows that a dam first gives by tension of vertical sections near the toe. (2) An accepted form of cross-section is shown to be stable as far as horizontal sections are concerned, but unstable by applying same conditions of stability to vertical sections. (3) Distribution of shear over base must be more nearly parabolic than uniform, but as no reversal of statement follows in passing from former to latter extreme hypothesis, it is not unreasonable to assume the latter distribution will describe the facts fairly closely until we have greater knowledge. (4) Masonry dams must be investigated for stability of their vertical sections. If this be done, it is believed that most existing dams will be found to fail, if criteria of stability usually adopted for their horizontal sections be accepted. This failure can be met in two ways: (a) By modification of customary cross-section (probably section like Vyrnwy dam would give better results than more usual forms); (b) by frank admission that masonry, if carefully built, may be trusted to stand tensile stress. It is idle to assert that it is absolutely necessary that line of resistance shall lie in the middle third for a horizontal treatment, when it lies well outside middle third for at least half the dam for a vertical treatment.

Investigations by Sir Benj. Baker, Prof. W. C. Unwin and others have not supported the above contentions to such degree as to cause alarm. Experience with the large number of masonry dams, some of which have been in service many years, is reassuring.

Investigations by Unwin* on dams of various sections led to the following conclusions: (1) For rectangular dam, distribution of shearing stress on horizontal planes may be represented by ordinates of a parabola. (2) For triangular dam, distribution may be represented by ordinates of a triangle with apex beneath toe. (3) For dam with vertical back and curved front

* "On the Distribution of Shearing Stress in Masonry Dams," in Engineering (London), Vol. 79, 1905, p. 825

† The component displacements of a point parallel to 3 rectangular directions are termed the 3 shifts. (Todhunter and Pearson, "History of Elasticity.")

distribution may be represented by a figure consisting of a parabola superposed on a triangle. (4) If lower portion of profile of dam is a rectangle, distribution is represented by a parabola.

Sir John Walter Ottley and Arthur William Brightmore, in "Experimental Investigations of Stresses in Masonry Dams subjected to Water Pressure" (Proc. I. C. E., Vol. 162, 1905), concluded from experiments that: (1) If a masonry dam be designed on assumption that stresses on base are uniformly varying and that stresses are parallel to resultant force acting on base, actual normal and shearing stresses on both horizontal and vertical planes would be less than those provided for. (2) There can be no tension on any planes near toe. (3) There will be tension on certain planes other than horizontal near heel, maximum intensity of such tension in foundation being generally equal to average intensity of shearing stress on base, the inclination of its plane of action being about 45°, and its maximum intensity in the dam above base about half the above amount and acting on a plane less inclined to the horizontal.

CURVED OR ARCH DAMS

Adaptability. Several things affect the relative suitability of straight or arched form for the plan of a dam: (1) shape and nature of site; (2) magnitude of dam (especially length and hight); (3) convenience of construction; (4) simplicity and safety of design; (5) cost. At opposite extremes under (1) are broad sites with relatively gently sloping sides with rock under cover of earth, common in eastern U. S., and narrow canyons with approximately vertical bare rock sides found in the mountain regions of the West. For the former the arched dam is not suitable; for the latter its advantages are often obvious; for sites of less pronounced type only thorough study and sound judgment can reach proper selection. For very low dams the arched form is usually not advantageous; for dams of medium or great hight on sites having approximately vertical rock sides the arched dam may properly be designed of much thinner section for spans not exceeding, say, 600 ft., provided relation of radius and central angle are chosen so as to give the maximum probable arch resistance. There has been much difference of opinion on these questions and no generally accepted method of computation for arched dams exists. A few formulas are given below. Discussions, with other methods and formulas, will be found in T. A. S. C. E., Vol. 53, 1904, pp. 89–209; Trans. Tech. Soc. Pacific Coast, Vol. 6, 1889, p. 75.

Woodard's Formulas.* In an arched dam of relatively thick cross-section the load will be divided between arch and gravity action in proportion to their relative rigidities. Assume that line of thrust in arch coincides with center line throughout length. Neglect the effects of the fixed abutments which stiffen the arch and make it take more of the load, leaving less for the gravity section. Deflection will be uniformly increasing from the ends toward the middle.

A = sectional area of arch ring;
R = radius of upstream edge;

* Silas H. Woodard, "Lake Cheesman Dam and Reservoir." T. A. S. C E, Vol. 53, 1904, pp. 113–207.

E = modulus of elasticity of masonry;
F = compressive stress in arch ring;
α = one-fourth the angle which arch subtends;
L = length of center line of arch ring;
P = horizontal water pressure;
X = load borne by vertical beam;
D_a = deflection of crown of arch $= \dfrac{(P - X)RL \cot \alpha}{2AE}$ (1)

The deflection of the dam, considered as a vertical beam is:

$$D_n = \int_0^H \frac{MdM}{EIdX_n} dy \quad (2)$$

M = bending moment at any point;
dy = vertical distance between loads, dX_n;
I = moment of inertia of beam;
H = hight of dam = total length of beam;
D_n = deflection of any loaded point under load of dX_n from position of beam without loads, measured in direction of action of load X_n.

Here are two expressions for the same thing. By assuming successive arch rings, beginning at top of dam, each having a vertical thickness of 10 ft., and substituting in (1) arch deflections are derived; and by substituting in the approximate formula, $F = PR \div A$, stresses in the arch may be computed. For Cheesman dam, of curved gravity type, it was computed that the arch carried nearly half the load at the top, but only about 6 per cent. half-way down, and practically none at the bottom. Computation of stresses in the trapezoidal section included between two radial vertical planes, is then made by the method below, and the corresponding expressions for arch and beam are equated.

b = breadth of joint;
C_1 = width of upstream edge of joint;
C_2 = width of downstream edge of joint;
r = radius of downstream edge of joint;
s_1 = batter of upstream face in the 10 ft. next above joint;
s_2 = batter of downstream face in the 10 ft. next above joint;
a = area of horizontal joint $= \frac{1}{2}(C_1 + C_2)b$;
W = weight of all masonry above joint, in terms of weight of a cubic foot of masonry;
g = distance of center of gravity of joint from upstream edge $= \dfrac{b(C_1 + 2C_2)}{3(C_1 + C_2)}$; this holds for any trapezoid;
M = moment of all masonry above a joint about its upstream edge;
n = distance from upstream edge of a joint to resultant for reservoir empty $= M \div W$;
t = actual thrust of water against 10-ft. layer next above joint, in terms of weight of a cubic foot of masonry;

M_1 = moment about joint of all these horizontal loadings (X) which act above it;

v = distance between resultants for reservoir full and reservoir empty = $M_1 \div W$;

I = moment of inertia of joint about its center of gravity = $(C_1^2 + 4C_1C_2 + C_2^2)b^3 \div 36(C_1 + C_2) = \dfrac{(C_1^2 + 4C_1C_2 + C_2^2)}{72a} b^4$;

f_1 = vertical stress, in pounds per sq. ft. at upstream edge, reservoir empty $= \dfrac{156.25W}{a} + \dfrac{156.25W(g - n)g}{I}$;

f_2 = vertical stress, in pounds per sq. ft., at downstream edge, reservoir empty $= \dfrac{156.25W}{a} + \dfrac{156.25(g - n)(b - g)}{I}$;

f'_1 = vertical stress, in pounds per sq. ft., at upstream edge of joint, reservoir full $= \dfrac{156.25W}{a} - \dfrac{156.25W(n + v - g)g}{I}$;

f'_2 = vertical stress, in pounds per sq. ft., at downstream edge of joint, reservoir full $= \dfrac{156.25W}{a} + \dfrac{156.25W(n + v - g)(b - g)}{I}$.

With this treatment there is no question whether or not resultants fall within the middle third; that condition applies only when the horizontal sections are rectangular.

Provision for Weight (E. Sherman Gould).* The weight of the dam itself is not borne by the arch, and the pressure that the arch does sustain is uniform per unit length of surface and is directed at all points toward the center of the curve. The line of pressure, therefore, must correspond with the curve. Horizontal compressive stress per unit of section by Navier's formula is:

$$Q = PR'$$

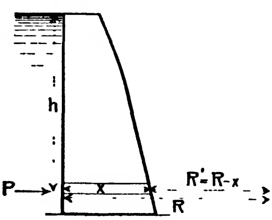


FIG. 48.—Vertical section of arched dam.

Q = compressive stress upon any horizontal strip, of length, x, and unit width, situated at depth h below surface of water, assumed level with top of dam. P = horizontal pressure at depth h upon unit surface. R is constant, while R' varies with value of x. To find proper value for x at any given depth h below top of dam, a value for Q, based upon the permissible unit stress L, must be assumed. Q would be this stress multiplied by x, since x, being of unit width, must resist this pressure.

$$x = \frac{62.50hR}{L + 62.50h}$$

A high value may be assigned to L, because the pressure, particularly near the base, is not upon imaginary isolated strips, as assumed in the formula, but upon strips embedded in a mass of masonry. L = 30,000 lb. per sq. ft. would be conservative.

* Trans. Am. Soc. C E., Vol. 53, 1904, p. 139.

Triangular Profile (Reuben Shirreffs).* This profile makes all the mathematics simpler; it is the theoretical form for a gravity dam as well as for an arched dam, if, in the latter case, arch elements carry all the load, and if it be further assumed (not far from the truth) that the average normal stress at all points is $F = PR \div A$, as on p. 136. For any new situation it will probably save time to use the triangular profile for first approximations, afterward making the necessary modifications to give the top a reasonable width. Instead of adding to the triangular form a parallel top, there should be added only a moderate top width, with a change of the batter for a considerable distance. If a considerable top width be added to the triangular profile, the result will be negative loads on the vertical beams toward the top, giving tension at the upper joints at the downstream face.

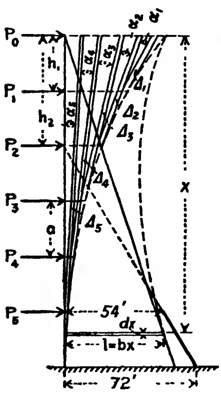


FIG. 49

Deflections, under concentrated load, of a beam having a varying moment of inertia but a constant thickness = 1.0. It is necessary that the beam be limited by straight lines between the load points.

l = joint length;
a = vertical distance between loads.
b = ratio of increase of joint length;
R = radius of upstream face of dam;
I = moment of inertia of joint;
h = hight
e = distance from neutral axis to extremity of joint;
E = modulus of elasticity of masonry;
M_1 = moment of all horizontal loads above joint considered;
δ = stress at extremity of joint;
λ = change of length of element, dx, of beam, due to δ;
ρ = radius of curvature of deflected beam;
x = distance below crest.
α = angle of deflected beam at any load point, n, having reference only to portion of beam between that load and one next below;
Δ = deflection of each individual portion of beam, produced by all loads above that portion;
D_n = total deflection of beam at any load point, n:

$$d\Delta = 12M_1\,(x - h)\,\frac{dx}{b^3Ex^3} \tag{1}$$

$$\text{and } d\alpha (\text{or also } d \tan \alpha) = \frac{12M_1dx}{b^3Ex^3} \tag{2}$$

After obtaining the values of Δ_n and $\tan \alpha_n$, these must be cumulated as follows for D_n:

$D_5 = \Delta_5$;
$D_4 = \Delta_5 + \Delta_4 + a \tan \alpha_5$;
$D_3 = \Delta_5 + \Delta_4 + \Delta_3 + a(2 \tan \alpha_5 + \tan \alpha_4)$.
and so on, as in Fig. 49.

* T. A S. C. E, Vol 53, 1904, p 155.

$$b^3E\Delta_5 = 12\int_5^1 P(Ln6a - Ln5a) - \frac{1}{30}(61P_1 + 62P_2 + 63P_3 + 64P_4 + 65P_5).$$

($Ln(a) = \log_e a =$ Naperian log (a)).

$$b^3E\Delta_4 = 12\int_4^1 P(Ln5a - Ln4a) - \frac{6}{100}(41P_1 + 42P_2 + 43P_3 + 44P_4).$$

$$b^3E\Delta_3 = 12\int_3^1 P(Ln4a - Ln3a) - \tfrac{1}{3}(25P_1 + 26P_2 + 27P_3).$$

$$b^3E\Delta_2 = 12\int_2^1 P(Ln3a - Ln2a) - \tfrac{1}{3}(13P_1 + 14P_2).$$

$$b^3E\Delta_1 = 12P_1(Ln2a - Lna) - 7.5P_2;$$

$$ab^3E\tan\alpha_5 = \frac{1}{150}(49P_1 + 38P_2 + 27P_3 + 16P_4 + 5P_5).$$

$$ab^3E\tan\alpha_4 = \frac{3}{200}(31P_1 + 22P_2 + 13P_3 + 4P_4).$$

$$ab^3E\tan\alpha_3 = \frac{1}{24}(17P_1 + 10P_2 + 3P_3).$$

$$ab^3E\tan\alpha_2 = \tfrac{1}{6}(7P_1 + 2P_2).$$

$$ab^3E\tan\alpha_1 = 1.5P_1.$$

In applying the foregoing method to a beam of irregular profile, but which can be bounded by straight lines between load points, it is only necessary to remember that for each beam section there will now be a different origin for x, and a different value for b. For example, if the bottom section of a beam have a joint length of 54 on top and 72 on bottom, with $a = 30$, origin of x will be at econd load point, P_2 (see Fig. 49), value of b will be 0.60, $x - h = x - 90$, and

$$M_5 = P_1(x + 30) + P_2x + P_3(x - 30) + P_4(x - 60) + P_5(x - 90).$$

It is next necessary to modify the results derived from the beam of constant thickness to suit the case of the beam comprised between radial planes 1 ft. apart at the upstream edge of the dam. Elementary deflections will vary inversely as moments of inertia of the rectangle having the hight l and base of 1.0 and a trapezoid having same hight and a base $= 1.0$, but a top $c_2 = 1 - \frac{l}{R}$. The ratio of these moments of inertia is

$$m = \frac{c_2^2 + 4c_2 + 1}{3(c_2 + 1)}$$

Multiply individual deflections and deflection angles by reciprocal of m obtained from the joint length at the middle of the section under consideration, before cumulating the individual results for deflections D.

For the triangular profile, the following simple formulas aid rapid computation. Let V be volume and W weight of a vertical slice of the dam between radial planes 1 ft. apart at the upstream face; g the distance of its center of gravity from same face; g–n the distance from the vertical through the center of gravity to the point of the base where the resultant pressure strikes; M' total moment of all loads above base; Δ the unit weight of masonry:

$$V = \frac{3R - bh}{6R} bh^2; \; W = \Delta V;$$

$$\frac{g}{l} = \frac{2R - bh}{6R - 2bh} \tag{3}$$

$$\frac{g - n}{l} = \frac{6M'}{\Delta\left(3 - \frac{bh}{R}\right) b^2 h^3} \tag{4}$$

Arch and Cantilever Method (Edwin Duryea, Jr.).* In a masonry dam any point, A, is deflected downstream as the water rises. In an arch-type dam displacement of A corresponds to the deflection of a cantilever beam of unit length between two transverse vertical sections, and to the deflection of an arch lamina of unit hight between two horizontal planes. The total water pressure will be shared between arch and cantilever, directly as respective rigidities or inversely as deflections. Cantilever beams are assumed fixed in direction at base, with planes before flexure still planes after flexure; arch laminæ are assumed to be two-hinged, or changeable in direction at each bank. These assumptions are contradictory, and both tend to reduce the apparent proportion of pressure carried by arch action.

1. Vertical section through A:

h = hight of dam from base (below A) to crest,
b = thickness of dam at base (below A),
x = hight from base of dam to any horizontal section,
l = thickness of dam at hight, x. $[= \frac{1}{2}(h - x)]$

2. Horizontal section through A:

s = span of horizontal arch lamina before being shortened by pressure,
r = mean radius of horizontal arch lamina before being shortened by pressure = $R - l/2$,
l_1 = chord of half of horizontal arch lamina before pressure is applied,
l_2 = chord of half of horizontal arch lamina after pressure is applied,
T = thrust on horizontal arch lamina due to proportion of water pressure borne by it,
e = total shortening of chord l_1 due to thrust T; y_1 and y_2 = ordinates corresponding to l_1 and l_2,
$D = y_1 - y_2$ = deflection of point A.

3. Water pressures:

γ = weight of a cubic foot of water = 62.5 lbs.,
γ_g = proportion of unit weight of water (number of 62.5ths) borne by gravity action,
γ_a = proportion of unit weight of water (number of 62.5ths) borne by arch action.

4. Compression of masonry:

E = modulus of elasticity = 1,500,000 lb. per sq. in.

$$\frac{h^3\gamma_g}{b^3 E}x^2 = r - \sqrt{r^2 - \frac{s^2}{4}} - \sqrt{\left[\frac{s^2}{4} + \left(r - \sqrt{r^2 - \frac{s^2}{4}}\right)^2\right] \times \left[1 - \frac{\gamma_a(h-x)}{lE}r\right]^2 - \frac{s^2}{4}}$$

* Trans. Am. Soc. C. E., Vol. 53, 1904, p. 172.

After selection of any particular arch lamina, the only unknown is γ_a. Seven-place logarithmic tables are not sufficiently accurate; compute by ordinary multiplication.

A formula for arch deflection by Vischer and Wagoner,* is much simpler:

$$e = DV$$

e = total shortening under arch pressure of half the arch lamina;
D = deflection of center point of arch lamina;
α = one-quarter of total angle (in radians) subtended by arch lamina.

From theory of compression of elastic bodies,

$$e = \frac{T}{lE}l_l, \qquad D = \frac{\gamma_a(h - x)r}{lE\alpha}l_l,$$

where the vertical thickness of the arch lamina is unity and l_l represents the half length of the arch lamina instead of the length of the half chord.

For a triangular profile with base = ½ hight, and water level with apex:

$$\gamma_a = \frac{125x^2}{r^2 + 2x^2}$$

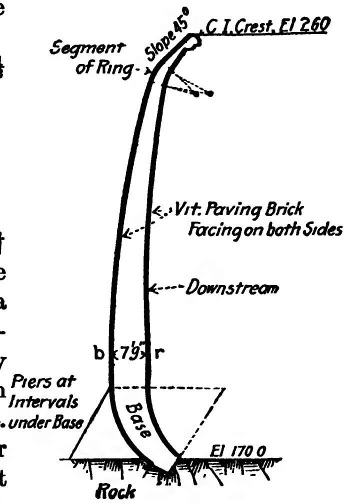


Fig. 50.—Ithaca Dam.

Ithaca Dome Dam (G. S. Williams).† Where a dam abuts against vertical walls the action becomes more nearly akin to that of a plate supported on three edges. One criticism against arch dams has been that, by reason of rigidity of the base, arch action could not be developed in their lower part. To overcome this objection and avoid so far as possible stresses of opposite signs acting at right angles to each other, recourse was had to a design similar to an egg-ended boiler, and the base was made in the form of a portion of a torus. Radius of vertical curve of the base was selected so that the upward thrust at junction with upper part of dam would never exceed the downward pressure transmitted through the material above. By this construction it became possible to utilize the bed of the stream as an abutment and still permit elastic deformations and true arch or dome action near or at the base. By inclining the radius at the junction of the torus with the superimposed cylinder, an upstream thrust at this point was obtained from the former, tending to oppose the pressure of the water and decrease the horizontal circumferential thrust in the cylinder. Similarly, the inclination of that portion of the structure above *br* also introduces a thrust upstream, acting likewise to decrease the horizontal circumferential thrust. Above this plane, to 10 ft. below crest, the section is made up of a series of frustums of conical shells.

* Vischer and Wagoner: "On the Strains in Curved Masonry Dams," Trans. Tech. Soc., Pacific Coast, Vol 6, Dec., 1889, pp 75–151.
† T. A. S. C. E., Vol. 53, 1904, p. 182.

Next 4 ft. is a segment of a ring, and remainder to the crest is a segment of a conical dome. Radii of intrados, or downstream face, vary from 57.75 ft. at bottom, to 58 ft. at mid-hight, to 50 ft. at crest. The maximum radius of the extrados is 67.75 ft., at crest, 51 ft. The axes of the two faces are not coincident, that of the downstream face being 2.25 ft. upstream from that of the upstream face, thus making the dam somewhat thicker at the abutments than at the center. Hight, 90 ft. Fig. 50 shows a section.

The shape of the crest was selected for the following reasons: To discharge a maximum quantity at heads above 2 ft.; to permit ice climbing it (slope of 45° answers well); to insure positive and continuous aeration of the region behind the sheet and the prevention of even a minute vacuum there; to deliver the overfalling sheet well away from the toe.

For preliminary purposes, $T = pR$, wherein T = thrust or pull of sheet, p = normal force, and R = radius of face to which force is applied. Were the sections cylindrical, p would be water pressure and R horizontal radius, and this formula would be rigidly applicable for determination of arch stresses. In the present design, the faces are generally inclined, and the formula becomes $T = pR \sec i$, i being angle of inclination of the upstream face from the vertical. If the thickness be F, then the unit thrust, $t = \dfrac{PR \sec i}{F}$, for a section one unit high, omitting the effect of the inclination in producing a radial thrust opposite to T. As this counterthrust actually reduces T, stresses computed by the formula will be greater than those really existing in the horizontal circumferential direction. Thrusts in the torus base, being largely absorbed by the vertical arch, give much lower unit stresses.

For final and more accurate determination: A vertical slice of the dam at the center, 1 in. thick at the upstream face, was cut by vertical radial planes and divided into blocks by planes normal to the upstream face. Beginning at the top, the forces due to water pressure and to the weight of the block above the plane of its base are combined by a simple triangle of forces and the resultant P resolved into a horizontal component, P_h, and one normal to the base, P_n. For the next section, this resultant P is combined with the weight of the added block and the force due to pressure upon it, and a new resultant obtained, which is resolved as above. The forces acting on the block in question are:

P_w = water pressure on face of block.
P_n = component of total pressure, P, normal to base.
P_{n-1} = component of total pressure, P, normal to plane of top of block, which = P_n for the section next above.
ΔG = weight of block.
i = angle which the normal to upstream face makes with horizon.
H = total horizontal force carried by horizontal arch.
F_v = area of vertical faces.
F_b = area of normal faces.
t = unit pressure, per sq. in., on horizontal arch at center of section.
s = unit pressure, per sq. in., on vertical arch at base of section.
ψ = angle between vertical sides of block;
θ = angle between top and bottom of block.

$$H = 2T \sin \frac{\psi}{2} = \left[P_w - (P_n + P_{n-1}) \sin \frac{\theta}{2}\right] \sec i + \Delta G \tan i$$

But, from dimensions of slice, $\sin \frac{\psi}{2} = \frac{0.0417}{R}$

therefore, $H = 2T \times 0.0417 \div R; T = \frac{HR}{0.0834}; t = T \div F = HR \div 0.0834F; s = P_n \div F_b.$

Unit stresses in the horizontal arches of the torus base are less for a flood than for full pond. Stresses near the crest are amply provided for by the crest casting and the steel channels in that portion.

Stresses at low water and when pond is empty: Examining horizontal thrusts due to the weight of the dam, it was found that for the segment of ring near top, outward thrusts were decreasing downward. Whence, in an ordinary dome, tension would occur in this region, and should be resisted, but any yielding in the haunches must be accompanied by a lowering of the crest at the center. Because of the rigidity of the abutments lowering will be resisted by the hyperbolic arch formed along the vertical plane through the crest. Consequently the hoop usually supplied to a dome at joint of rupture is not needed, although, to relieve small tensions which might occur while the vertical arch resistance was developing, a hoop of steel should be provided at the haunch. At the top of the torus base, similarly, tension would occur with the pond empty, were it not that piers under the heel support the upper masonry. These piers have no bond with the body of the dam, which is free to move away when loaded; they act simply as wedges to keep the structure erect when there is no pressure on the back, and prevent tensile stresses at the top of the torus base.

Baum arch dam is adapted to narrow canyons in solid rock. The main feature and the one claimed to make economy of construction feasible is the keeping of the subtended angle of the arch as nearly constant as possible, and as near to the most economical value at any elevation, as the site will permit. This necessitates the abandonment of the constant upstream radius generally employed, and the substitution of radii of varying length determined by the width of the canyon at any elevation.

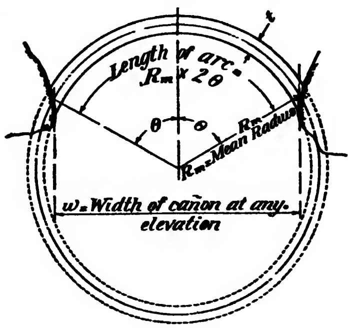


FIG. 51.

Assume as usual that any unit horizontal element, such as shown in Fig. 51, is a portion of a cylindrical ring, in which the average stress q, in pounds per sq. ft. is equal to the radial load P (lb. per sq. ft.) multiplied by the radius of the upstream face and divided by the area of the section:

$$q = P(R + t) \div A \tag{1}$$

t = thickness of the dam; R = radius of downstream face; A = area of section at any given point.

The volume of masonry is equal to the area of the section, times the mean radius, times the enclosed angle.

$$V = A \times R_m \times 2\theta \tag{2}$$

$$V = \frac{C' \times \left(\frac{\omega}{2}\right)^2 \times 2\theta}{\sin^2\theta} = \frac{K\theta}{\sin^2\theta} \tag{3}$$

in which C' and K are constants, the latter depending upon the width of the canyon.

From Fig. 52, amount of masonry for an arched dam will be a minimum when the mean radius at any elevation is so chosen that 2θ is about 133°; the variation in masonry required for a given dam will be only about 1 per cent., provided that 2θ be held between 120° and 146°. This method involves the independent determination of the dimensions of the successive arch-shaped

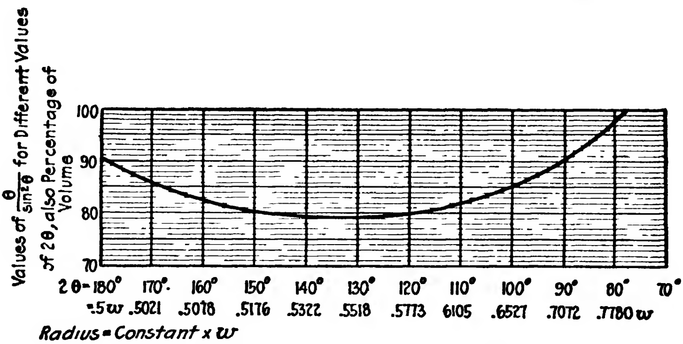


FIG. 52.—Baum arch dam. Relation of 2θ to $\theta \div \sin^2\theta$.

slices between predetermined levels; each being considered primarily as an independent structure; such slices being thereafter superposed. At the top of the dam it will generally be best to choose 2θ near the upper limit (146°) for greatest economy, and at the bottom near the lower limit (120°). Slices of dam have no common center line; centers are located principally to get the length of arch as short as possible for a given distance across the canyon. So far only the average stress q has been considered. Toward the bottom it is necessary to investigate also the maximum stress. The maximum arch compression will be $q \times \dfrac{2(R+t)}{2R + \frac{3}{2}t}$ and will exist along the downstream edge. The stress on the foundation does not need to be considered for dams of this type less than 200 ft. high. This method can also be applied to the multiple-arch dam; the most economical result will be obtained by taking the top width equal to the distance between centers of buttresses, and bottom width equal to the distance between outside of buttresses, and by choosing the enclosed angle less than the most economical at the top, where excess thickness must be provided for mechanical reasons, and gradually increasing this angle toward the foundation.

Having calculated the radii and thicknesses of several horizontal slices, the top thickness, which is generally chosen, is set off and arcs with radii R_6 and

$(R_6 + t_6)$ are drawn on the contour map (Fig. 53); the centers of such circles will be on a perpendicular to the center of chord W_6. Thickness t_5 is then set off, arcs with radii R_5 and $(R_5 + t_5)$ drawn to intersect contour V, bearing in mind that the center common to these latter circles does not need to be on the perpendicular to chord W_6. It is usually preferable to assume the downstream faces of the upper slices vertical adjacent to the center, at least for the first

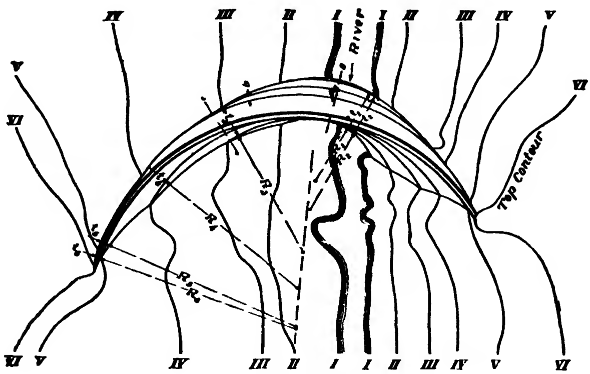


Fig. 53.

trial; although after the lower slices have been laid in, it may be found desirable to slope the central portion of such face one way or other. It has been found that a vertical wall in this part usually affords the most economical construction. In other parts, the downstream face will always have some slope, due to method of design. The procedure above outlined is followed to get up- and downstream face lines at lower elevations. Discretion must be exercised in locating centers of arch slices, especially where a very abrupt change occurs at one side only of the canyon. While generally preferable so to form

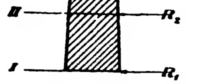


Fig. 54.

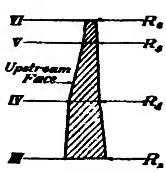


Fig. 55.

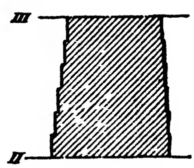
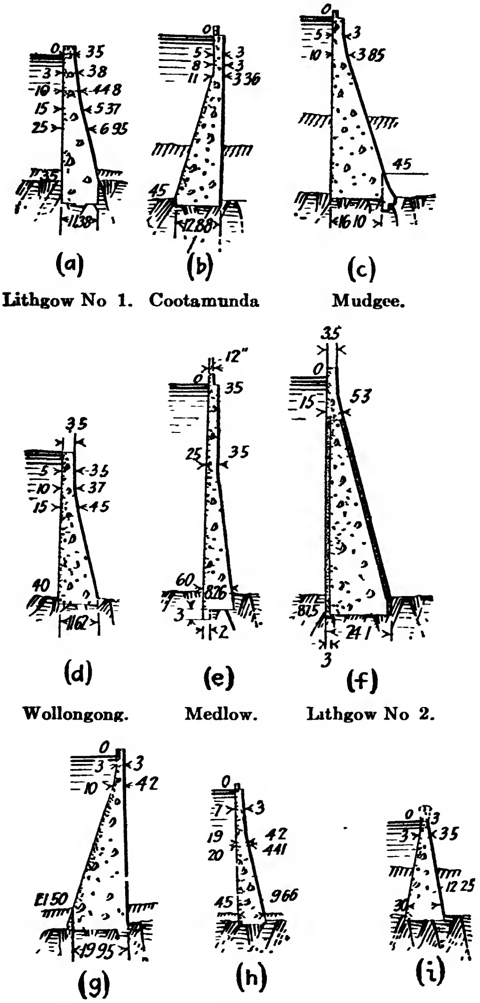


Fig. 56.

slices between selected elevations that the tops and bottoms of adjoining slices may be regarded as being superposed in strict coincidence, as shown in Figs. 54 and 55, it is desirable in some localities to face the dam with cut stone, and it may be of advantage in such case to step the faces as shown in Fig. 56.

As the dam must also be safe with reservoir empty, it is necessary to have the thickness increase from the crest to the foundation, to prevent overhang. The proportional increase in water pressure must, therefore, be greater than

the proportional decrease in length of the upstream radius toward the foundation. The ratio of increase in water pressure is always fixed and the ratio of decrease in length of the upstream radius depends upon the slope of the canyon sides. Now, if slopes are such that at any intermediate elevation the ratio of decrease in length of the upstream radius (corresponding to a 133° arch) has



(a) Lithgow No 1. (b) Cootamundra (c) Mudgee.

(d) Wollongong. (e) Medlow. (f) Lithgow No 2.

(g) Tamworth. (h) Wellington. (i) Parkes.

FIG. 57.—Sections of curved masonry dams. Dimensions in feet.

been greater than the ratio of increase of water pressure, a decrease in the thickness of the dam at this elevation would result and the structure would be overhanging. If a certain thickness must be provided to prevent overhanging, it is most economical to throw normal load on the total area by increasing the length of the upstream radius above that corresponding to a 133° arch for the reason that a flat arch requires less material. By flattening the arch, the

enclosed angle will have to be varied between 140° and 60° to get the most economical dam, and one which also will satisfy requirements for safety with no water.

The main features in this type of dam have been patented by F. G. Baum & Co., San Francisco, Cal. (See "The Constant-angle Arch Dam" by L. R. Jorgensen, Trans. Am. Soc. C. E., Vol. 78, 1915, p. 685.)

Table 43. Details of Some of the Principal Curved Masonry Dams

Arranged in order of hight. See sections on pp. 146 and 191.

Dimensions of following curved dams will be found on pp. 166 to 191: Arrowrock, Barren Jack, Beetaloo, Burraga, Butte, Chartrain, Chemnitz, Furens, Hemet, Gileppe, Granite Springs, Hijar, Roosevelt, San Mateo, Thirlmere, Turlock, Upper Otay, Urft, Villar, Wigwam, Zola

Section of Halligan dam, p. 191.

Place	Max hight above foundation, ft.	Base thickness, ft.	Top thickness: Thickness, ft	Top thickness: Depth below crest where measured, ft.	Surcharge allowed ft. of water	Max. arch stress, lb. per sq in.	Radius, upstream face, ft.	Length, ft.	Rock in foundaiton	Date
(a) Katoomba, N S W .	25 0	20 29	3 0	0	1 0	233	220 0	320	Sandstone .	1905
(a) Picton, N S W ...	28 0	13 62	7 0	0	10 0	186	120 0	112	Sandstone	1897
(c) Winchester, Ky	31 0	8 58	4 83	0	— —	498	318 4	407	— — — — — — — —	— —
(a) Queen Charlotte Vale, N S W.	32 0	8.65	3 0	0	— —	155	90 0	113	Quartzite	1898
(a) Parkes, N S W......	33 5	13 5	3 0	6 0	5 0	373	300 0	540	Granite	1897
(a) Lithgow, No. 1, N S W.	35 0	10.88	3 5	3 5	3.5	155	100 0	178	Sandstone	1896
(a) Wollongong, N S.W	42 0	11 62	3 5	5 0	1.0	311	200 0	535	Basalt.	1898
(a) Cootamunda, N S W.	46 0	13 0	3 0	8 0	1.0	389	250 0	640	Granite	
(a) Wellington, N S.W..	48 0	10 0	3 0	7 0	2 0	311	150 0		Conglomerate	1899
(a) Mudgee, N S W....	50 0	18 0	3 0	5 0	1 0	311	253 0		Altered slate	1899
Las Vegas, N M....	50 0	15 5	5 0	0	— —	350	250 0	.210	Sandstone	1910
(a) Parramatta, N S W.	52 0	15 0	4 8	0	2 0	223	160 0	225	Sandstone	— —
(a) Lewiston, Idaho. .	55 5	14 5	5 33	0	— —	475	286 5	288	— — — — — — — —	— —
(a) Tamworth, N S W.	61 0	21 5	3 0	3 0	2 0	311	250 0	440	Granite .	1898
(b) Bear Valley, Cal *.	64 0	8 4(h)	2 75	0	— —	825	335 0	300	Granite ...	1884
(a) Medlow, N S W...	65 0	8 96	3 5	21 0	3 0	186	60 0	124	Sandstone...	1906
(a) Lithgow, No. 2, N S W.	87 0	24 0	3 0	3 0	3 0	155	100 0	221	Sandstone...	1906
(d) Ithaca, N. Y. ...(f)	90 0	7 75	— — —	0	— —	283	67 85	— —	Shale	1903
(b) Sweetwater, Cal ..	94 0	46 0	12 0	0	— —	188	222 0	380	Porphyry	1888
Las Vegas (proposed)	95 0	43 3	5 0	0	— —	300	250 0	390	— — — — — — — —	— —
(n) Columbus, O. (Scioto R)	95(m)	64 3	11 0	0	0	— —	800 0	1806	Limestone	1905
(b) Barossa, S. Australia	113 0	34 0	4 5	0	— —	242	200 0	470	Shale	1903
(b) Zola, France . ..	123 2	41 82	19 02	0	— —	196	157 0	205	— — — — — — — —	1843
(d) Lake Cheesman, Col	218 5	176 0	18 0	0	0	200	400 0	710	Granite	1900
(e) Pathfinder, Wyo. .	210 0	72 66(k)	10 0	0	— —	181	150 0	425	Red granite	1909
Shoshone, Wyo.	328 0	108 0	10 0	0	0	— —	150(l)	175	Granite	1910

Principally from E. R, Oct. 8, 1910, p 403. (a) E N, May 19, 1910, also Proc. Inst. C E, 1909. (b) Wegmann's "The Design and Construction of Dams" (c) Data from Wm. Wheeler. Base thickness at 15 ft. above base (d) Trans. Am Soc C. E, Vol 53, June, 1904. (e) Jour N. E. W. W. Ass'n, June, 1906. (f) Actually built but 30 ft. high, stresses discussed on p 141. (h) 48 ft. below crest. (k) 94 ft below crest. (l) Radius of center, both faces battered. (m) Temporarily finished to a hight 22 ft less. (n) T. A S. C E, Vol. 67, 1910.

* Old dam; for multiple-arch reinforced concrete dam, built in 1911, see p 171.

MULTIPLE-ARCH, BUTTRESSED AND HOLLOW DAMS

Attempts to reduce cost and gain other advantages have been made by designing masonry dams with buttresses or pier walls, spaced apart a number of times their thickness, and spanning the spaces with thin arches (multiple-arch dams), walls, or decks. Under usual conditions affecting costs, the increase in cost per unit counterbalances the saving in quantity, wholly or partially. Excepting reinforced concrete dams, but few of this type have been built. Meer Alum dam, in India, and Belubula dam in New South Wales are examples.

Buttressed Dams. Geo. L. Dillman (in T. A. S. C. E., Vol. 49, p. 94, 1902) gives following formulas for dams having triangular buttresses with horizontal parabolic arches between. Theoretical shape is shown in Fig. 58. Practically, tops of buttresses will have width sufficient for footway or road, carried by vertical arches. Backs of buttresses and lines of parabolic arch vertices are

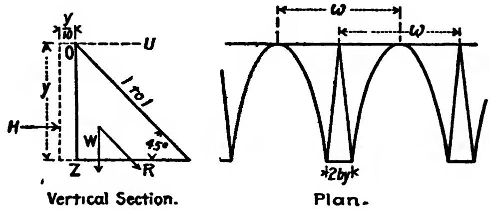


FIG. 58.

vertical; *i.e.*, upstream face of dam is plumb, but may be battered, with economy.

O = origin of co-ordinates.
w = length of unit of dam, center to center of buttresses, or between lines of vertices of arches.
W = total weight of masonry, @ $2\frac{1}{3} \times 62.5$ lb. per cu. ft.
y = depth from crest, or hight of dam.
H = horizontal thrust of water between buttresses $= \frac{1}{2}wy^2$ (in. cu. ft. of water).
b = batter of buttresses; $2by$ = width of buttress at toe.
A = area of horizontal section of buttress.
V = volume of buttress from crest.
x = distance of c.g. of A from vertical back of buttress, OZ.
n = distance of c.g. of V from vertical back of buttress, OZ.
m = distance of c.g. of face and buttress horizontally from OZ.
d = distance of c.g. of face and buttress from heel.
z and u = rectangular co-ordinates for R (z is positive, *downward*).
R = resultant of forces, reservoir full.

$$A = \frac{y}{3}(w + 4by). \tag{1}$$

$$V = \frac{y^2}{18}(3w+8by). \tag{2}$$

$$x = \frac{3y(w + 8by).}{10(w + 4by).} \tag{3}$$

$$n = \frac{3y(w + 6by).}{5(3w + 8by).} \tag{4}$$

Adding to buttress (dotted lines at left of Fig. 58) a "face," of uniform thickness $\frac{y}{10}$, its volume is $\frac{wy^2}{10}$; its c.g. is $\frac{y}{20}$ upstream from OZ.

$$m = 3y(17w + 120by) \div 160(3w + 5by). \tag{5}$$
$$d = 11y(9w + 40by) \div 160(3w + 5by). \tag{6}$$
$$W = 28y^2(3w + 5by) \div 135 \text{ (in cu. ft. of water)}. \tag{7}$$

Equation for R is:

$$z = \frac{56u}{135w}(3w + 5by) + \frac{y}{900w}(481w - 840by). \tag{8}$$

With $z = y$, R intersects base at:

$$u = y(1257w + 2520by) \div 1120(3w + 5by), \text{ at a distance} \tag{9}$$

$$\text{from toe} = y(2103w + 3080by) \div 1120(3w + 5by). \tag{10}$$

With reservoir empty maximum intensity of pressure at heel is:

$$q = \frac{7y}{3267w}(759w + 440by). \tag{11}$$

With reservoir full maximum intensity of pressure at toe is:

$$p = (1083w + 3080by) \div 6534b. \tag{12}$$

With weights and pressures in cubic feet of water, substituting chosen values for p and q, formulas (11) and (12) give values of $w \div b$; w should be assumed to make a good proportion with the hight of dam; then compute b. It is granted that shape suggested is not most economical of its type, but increased factors of safety and better chance for inspection and repair, as compared with "standard types" of solid dams, are claimed. Concrete is most suitable material.

Concrete Hollow Dams. A hollow reinforced concrete retaining dam may have its deck and top made up of segmental barrel arches, supported by pier walls or buttresses. To minimize effects of temperature, etc., the spaces between pier walls may be filled and the deck partially or wholly covered with gravel or other suitable material. Ambursen Co., Boston, Mass., has specialized in the construction of this type. Notable Ambursen dams are: Guayabal dam, Porto Rica, 115 ft. high, 1900 ft. long, built in 1913; La Prele dam, Wyoming, 135 ft. high. 300 ft. long, built in 1908.

Design of Buttressed Dams of Reinforced Concrete (R. C. Beardsley).* Uncertainty, other than the crushing strength of concrete, is practically eliminated. The engineer has only to compute strengths of beams and columns. The buttressed dam owes its simplicity to the fact that its upstream face is so flat in slope that the water pressure has a vertical component sufficiently in excess of the horizontal to hold the structure down against forces acting to overturn it or to slide it bodily downstream. In designing, the dead-load pressure upon the foundation should be determined first. Find the center of gravity of the weight areas. To do this buttresses and deck have to be considered separately. First, take the buttress (Fig. 59). To get the center of gravity:

$$z = \frac{L'}{4} \times \frac{3d + b}{2d + b} \qquad y = \frac{H'}{2} \times \frac{d + b}{2d + b}.$$

As a check on the calculation, the center of gravity must be on the line drawn from C' to the middle of the base. To get cubic contents of buttress $A'BC'$, find average thickness.

$$t = (2d + b) \div 3$$

The deck is a trapezoid, center of gravity of which is found (see p. 131) at G'. The resultant of two weights G and G' is found to act at G''.

* E. N., Apr. 23, 1908.

To determine the effect of the water vertically above the dam, construct the trapezoid of pressure $AEDC$ such that

$EA = (H + h)62.5 \div 145.$ 145 = weight of concrete per cu. ft.

$DC = h \times 62.5 \div 145.$

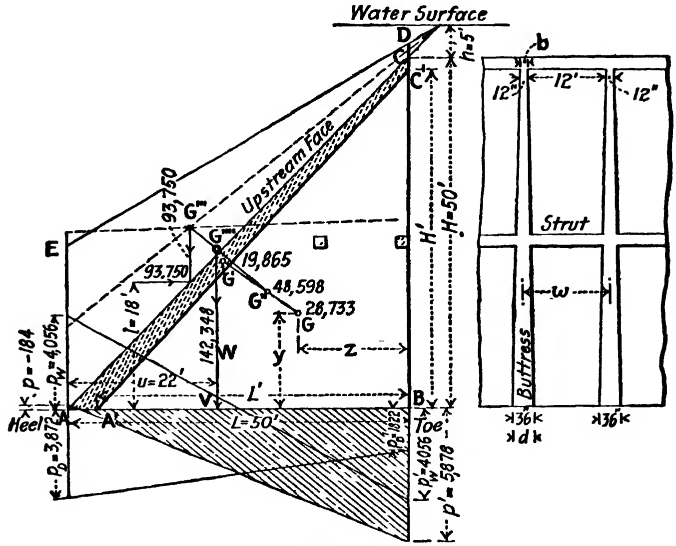


Fig. 59.

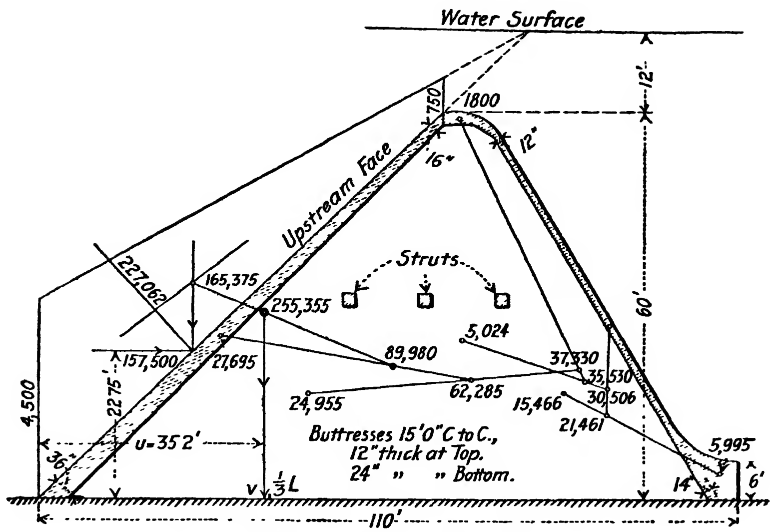


Fig. 60.—60-ft. buttressed dam with apron.
(Unless shown otherwise, numbers represent loads in pounds)

Distances are laid off to same scale as used in drawing the dam. Center of gravity of this trapezoid is at G'''. Weight of water vertically over the dam and carried by the dam is equal to area of trapezoid multiplied by 145. Result-

ant center of gravity of G'' and G''' is at G'''', where sum of all weights (buttress, deck and water), is applied. A vertical dropped to base of dam from G'''' cuts base at V, whose distance from A will be denoted by u.

For calculating pressures on foundation, two conditions are considered: (1) when $u = L/3$; (2) when u exceeds $L/3$. The third possible condition, u less than $L/3$, never occurs. The deck is always built at an angle not exceeding 45°, and when an apron on the downstream side is provided it is not given a flatter slope than shown in Fig. 60, where $u = L/3$. As the deck is made flatter and the apron steeper, the center of gravity moves toward the center of the base, and therefore in practice u is never less than $L/3$.

When $u = L \div 3$, $p_d = 2W \div L$

When $u > L \div 3$, $p_d = \dfrac{2W}{L}\left(2 - \dfrac{3u}{L}\right)$.

p_d = vertical pressure at that edge of foundation which is nearest the line of action of resultant W; u is to be measured from same edge. Vertical pressure at opposite edge is in any case:

$$p'_d = (2W \div L) - p_d.$$

In the case of first formula the diagram of pressures is a triangle, and $P'_d = 0$. Ordinarily, p_d will be the pressure at the heel, as in Fig. 59. Under horizontal water pressure, the dam acts as a cantilever. Distance of point of application of P above base, l in Fig. 61, is found as shown there, or as in Fig. 59. From general formula for beams, $M = SR$, where M is bending moment, S, pressure at extreme fiber and R, section modulus. Here $M = Pl$, $S = p_w$, and $R = bL^2 \div 6$, where b is width of section under consideration, here taken as 1 ft., so that (all dimensions being in feet), $R = L^2 \div 6$; also $P = \frac{1}{2}(H + 2h) \times 62.5\, H$.

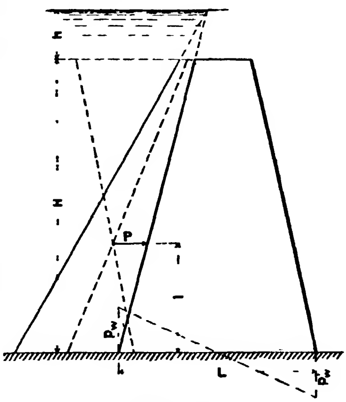


FIG. 61.

The resultant pressures due to both vertical loads and horizontal thrust are: $p' = p'_d + p'_w$ and $p = p_d - p_w$, down and upstream respectively.

These pressures are per foot of length of dam. However, as they are actually concentrated on the area covered by the buttresses, the pressures at bases of buttresses will be larger in the ratio of distance center to center of buttresses to width of base of each buttress. As a rule, 30,000 lb. per sq. ft. for p should not be exceeded unless the concrete is unusually well reinforced and 20,000 would represent the minimum stress to be used. If footings are set in trenches cut in solid rock, higher pressures may be used than would be proper where they merely rest upon the natural rock surface. If the base cannot take tension or may not safely be assumed to have tensile strength, method is inapplicable in all cases where it shows a resultant tension at one edge. In these cases it gives too small a value for compression at toe.

Negative pressure, or tension, at the upstream edge may be neglected, as

a cut-off wall is constructed along this edge, which is so thoroughly anchored to the rock along its whole length that lifting tendency is more than counteracted. The design of a dam with apron is carried out in the same way, the calculations being only slightly more complicated by the additional sections, centers of gravity of which must be found.

It can be shown that pressures are apt to be much larger in the case of the gravity dam without an apron than in the case where there is one, owing to the wider base made by the apron. When the dam exceeds about 30 ft. in hight, struts will have to be provided to prevent the tall, thin buttresses from buckling. An empirical rule is to place these struts at vertical distances of twenty-four times the thickness of the buttress, and horizontally about ten thicknesses apart. Struts should be reinforced the same as columns, but with enough additional steel on the under side for beam action to carry the dead weight of the concrete.

DAMS AND WEIRS ON POROUS FOUNDATIONS*

Length of Percolation. The safety of a dam founded on a porous stratum from being undermined by a gradual washing out of sand is dependent on the length of the enforced percolation. This length, L, should be sufficient to reduce the velocity of the undercurrent so that it will not disintegrate the sand; L must be some multiple of the head, H, or $L = cH$. Reliable values of the "percolation factor," c, suitable for different classes of sand, can be deduced only from experience. The following factors are based on experimental statistics.

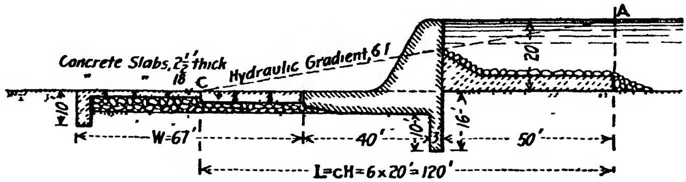


FIG. 62.—Granite Reef weir, Salt River project, U. S. Reclamation Service, as built.

Table 44. Values of Percolation Factor for Dams on Porous Foundations

Material	*Class*	*c*
Fine silt and sand, as in Nile river	A	18
Fine micaceous sand as in Colorado and Himalayan rivers. .	B	15
Ordinary coarse sand..	C	12
Gravel and sand	D	9
Bowlders, gravel and sand	E	4 to 6

Suppose the length of enforced percolation (the base of the dam), is insufficient and the hydraulic gradient too steep, the velocity of the percolating water will be capable of conveying away particles of sand from beneath the dam. Assume the sand of Class C, so that $c = 12$; with $H = 20$ ft., the required base length will be 240 ft. To increase the length of L, either (1) lengthen the base of the dam downstream; if an embankment it would have

* W. G. Bligh, E. N., Dec. 29, 1910 and Apr. 13, 1911. See also "Designing an Earth Dam," J. B. Hays, Proc. A. S. C. E., March, 1916.

to be enlarged by building to a slope from the new toe to the top; or (2) lengthen the impervious base rearward; in this case a comparatively thin impervious apron is sufficient.

Increasing the Length of Percolation. An efficient natural rear apron for dams across streams carrying silt is often formed by deposit; this should not be depended on, since, if a water-tight connection is not formed with the dam, or if a break occurs in the continuity of the apron, the incidence of the head will not be at the assumed upstream edge of the apron, but at some intermediate point, and so the effective length of percolation will be reduced below the safe limit. Another means whereby travel of percolation can be increased is a row of sheet-piling or some other impervious curtain wall. It has been ascertained experimentally that the value in increase of length of percolation due to a vertical obstruction is double that beneath a horizontal apron, for the percolating current travels down on one side of the obstruction, and up the other. This is subject to the proviso that vertical curtain walls, if more than one, are not spaced nearer than twice their depth. If the depth of the curtain is made equal to c in feet and the rear apron is reduced in length by $2c$, the length of percolation remains the same as before. The hydraulic grade will not be continuous but will have a step at the curtain, the depth of which will be equal to the neutralization of head effected by the curtain.

A diversion weir across a river consists of two main parts: the weir or drop wall, and the apron. As the hight of the weir seldom greatly exceeds

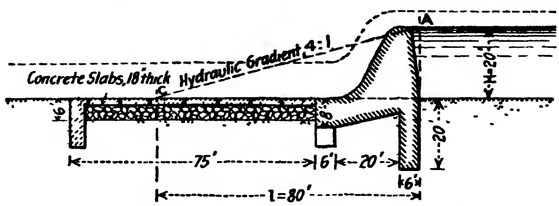


FIG. 63.—Granite Reef weir, as proposed by Mr. Bligh.

10 ft., except on a boulder bed, the weir itself is of minor consequence, the design being mainly centered on the apron. A fore apron is obligatory for protection of the bed from scouring; it should be of minimum width, W, measured downstream from dam, from the following formula based on actual practice:

$$W = 4c\sqrt{H_1 \div 13}$$

H_1 = the head above the upstream apron. The width of the fore apron, obtained by this formula, is in terms of the percolation factor, as the value of c is representative of the nature of the river sand. The lighter the sand, the easier will be the disintegration; consequently the longer will be the apron required. The formula is based on the hypothesis that with H_1 = 13 ft., the correct length for the fore apron will be $4c$. It is clearly of advantage to stop the fore apron at a width W, and make up the required length of enforced percolation by inserting an upstream apron. This substitutes clay, usually inexpensive, for masonry and the lifting pressure on the drop wall and fore apron is largely reduced. Beyond the impervious fore apron, riprap is neces-

sary, for a width dependent on two considerations: (1) hight, H_2, of the permanent obstruction, or drop wall, above low water; (2) the intensity of action of floods, which can be gaged by the unit discharge, q, over the weir. Taking the Narrora weir, Fig. 64, as model, width of riprap L_r will vary as $\sqrt{H_2}$, *i.e.*, $\sqrt{10}$, and also as the square root of q, *i.e.*, $\sqrt{75}$. $L_r = 10c\sqrt{H_2 \div 10} \times \sqrt{q \div 75}$.

Suppose $H_2 = 12$ ft., sand is of Class C, and $q = 150$ cu. ft. per sec., the width of fore apron will be $W = 4 \times 12 \times \sqrt{12 \div 13} = 48$ ft. $L_r = 10 \times 12 \times \sqrt{12 \div 10} \times \sqrt{150 \div 75} = 185$ ft. With a weir 40 ft. high on a bowlder bed, having $c = 5$, and $q = 215$ cu. ft. per sec., $W = 35$ ft., and $L_r = 173$ ft.

Construction of Aprons. W plus width of drop wall being never sufficient for percolation, the required value of L will have to be made up by a rear apron, or vertical curtain, or both. As 1 ft. of head is neutralized in every length equal to the percolation factor, c, it is convenient to have all longitudinal dimensions of the apron multiples of c. The thickness t of the fore apron must be such that the effective weight exceeds upward hydrostatic pressure. Weight can be represented by thickness times specific gravity, or $t \times \gamma$. At the same time the opposing hydrostatic pressure is represented by $(H - h)$, $h = L/c$ being the head neutralized up to point in question. In both expressions, unit weight of water is omitted, as a common factor. When the apron is immersed below low water, its weight will be $t\,(\gamma - 1)$. The weight should be one-third in excess of the actual requirements and the thickness will then be $t = 4(H - h) \div 3(\gamma - 1)$. The fore apron should taper toward the end to correspond with the progressive diminution of the water pressure, with 3 ft. as a minimum. The rear apron is made 5 ft. thick, following precedent, and is composed of wet clay overlaid by riprap. Riprap beyond the fore apron may be 4 ft. thick; in many cases, it is 5 ft. Any extension of the impervious fore apron beyond actual necessities is to be deprecated as increasing hydrostatic pressure on the work. On bowlder bed, the fore apron can be made pervious by leaving spaces between the blocks of concrete of which the floor can be built. This will necessitate a corresponding increase in length of rear apron to compensate for reduction in length of enforced percolation, so that it is of doubtful utility. With sand foundation such construction would not be practicable, as sand would be carried through the open spaces. In some weirs the fore apron is raised above low water level to avoid wet construction; riprap then will slope down to that level or below.

Shutters. Most weir crests are provided with collapsible crest shutters, often 3 to 6 ft. deep. This enables the permanent crest to be kept low, offering less obstruction to floods. Shutters increase the static head, maximum being when upstream water is at shutter crest, downstream channel empty.

Terminal curtain is required for protection in case of failure of the riprap, for which purpose it could well be made pervious, so as not to affect the hydrostatic status behind it. (Sometimes called "cut-off".)

Narrora Weir. Fig. 64 is a section of Narrora weir on the Ganges, as

originally constructed. Hydraulic gradient is 1 on 11, or probably steeper owing to the fore curtain, of circular undersunk wells connected with piling, not being absolutely impervious; the work stood for 20 yrs., but was always in unstable equilibrium. The hydrostatic pressure area shows a great deficiency in floor area, which must have been in high tension. Shortly before the final catastrophe, borings revealed the fact that the floor was then entirely

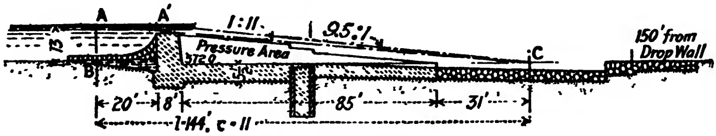


FIG. 64.

undermined and was only supported against collapse by hydrostatic pressure. After a heavy freshet in which the rear apron was washed out, the hydraulic gradient fell to 1 on 9.5 and the whole floor was blown up. Repairs (Fig. 65) lengthened the rear apron to 80 ft.; weir floor was built thinner, of heavier material, and strengthened by mass of concrete above original floor level; the length of impervious fore apron was reduced. Percolation factor, *c*, is increased in the new section to 15.7.

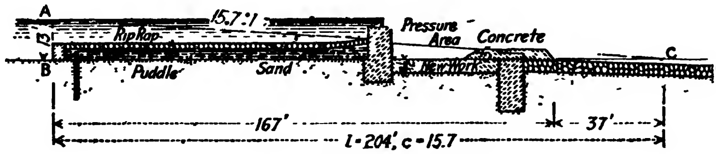


FIG. 65.

Chenab river weir was built on the same class of sand as in preceding example. After existing in insecurity for years, floor finally collapsed by undermining. The hydraulic gradient, on failure, was as low as 1 on 8.3, and the present value is 1 on 16.

Aprons of Egyptian Barrages. Fig. 66 shows the apron built for strengthening the old Nile Delta Barrage, or open dam. By means of this rear apron,

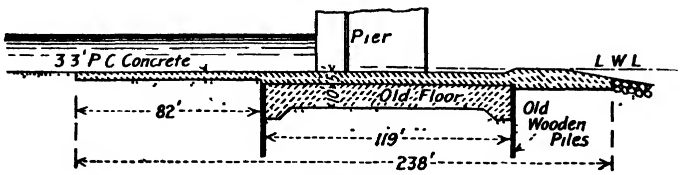


FIG. 66.

and the cement grouting, under pressure, of the old floor, the barrage was enabled to uphold a head of 14 ft., whereas previous to construction of apron, the work was useless. The value of L in Fig. 66 is horizontal, 238, vertical, 50, total 288 ft.; percolation factor will therefore be 288 ÷ 14, or nearly 20. In the Zifta Regulator, Fig. 67, the horizontal component is 149, and the verti-

cal, including everything, 66 giving a total of 215 ft. The head being 13.1 ft., c is 215 ÷ 13.1 = 16.4. In the Assiut Regulator, horizontal component is 169, vertical, 72, total, 241 ft. With a head of 11.5 ft., c = 21.

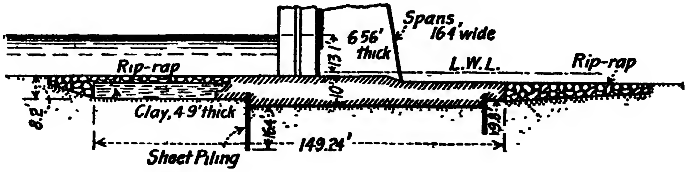


FIG. 67.

Pervious Floors. Theory developed for dams with impervious floors on porous foundations likewise applies to pervious floors. Many dams in East have floors of uncemented rubble, among them being Okhla weir, on Jumna river (Fig. 68), and Dehri weir, on Son river, India; Madaya weir, Madaya river, Burmah. Masonry cut-off walls are provided to prevent displacement of rubble.

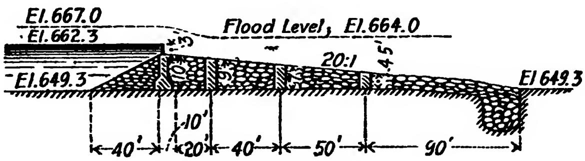


FIG. 68.

Sheet-pile Dams.* *Failure of Sheet-pile Dams.* Failures have generally occurred in spillway or overflow type, or diversion weirs. Seepage under sheet-piling tends to buoy up the particles of sand on the downstream side, while the surface of the river bed is frequently subjected to scouring action of water spilling over the crest. These forces combined move the finer particles first, then upward flow increases, and the destruction of the dam is in progress. Some engineers make the length of sheet-piling such that penetration below stream bed will approximately equal the head of water impounded. Many failures attributed to "unknown" causes were due to inadequacy of cut-off, often on account of careless driving and faulty alinement rather than insufficient penetration.

Length of Sheet-piling Required. H = maximum head of water on upstream side; D = depth of penetration for sheet-piling; s = specific gravity of material penetrated; p = proportion of solids. For average sand, $s = 2.65$ and $p = 1 - 0.4 = 0.6$. Then $D = H \div (s \times p) = 0.63H$ = theoretic penetration, or that at which excess hydrostatic pressure on upstream side is theoretically counterbalanced by weight of material on downstream side, of such size and porosity that the combined effect of friction and capillary attraction is a maximum. A length of sheet-piling which provides for penetration equal to maximum head at flood-stage, therefore, involves a factor of safety of only 1.59 (with a tight wall of sheet-piling in perfect alinement) applied to the

* Arnold C. König, Trans. Am. Soc. C. E., Vol. 73, 1911, p. 175.

theoretic minimum, manifestly inadequate for variable and uncertain conditions likely to be encountered.

Empirical Rules for Dams on Sand Foundation:
$D = 2H$ for heads up to 8 ft.;
$D = H + 5$ for heads up to 25 ft.;
$D = 0.66H$ as minimum in any case.

Special Features of Construction. The horizontal distance seepage water must flow is important in retarding its force, so that there will be no erosion. An apron serves, not only to resist the shock and abrasion of falling water, but also to seal the stream bed so that seepage under sheet-piling must flow diagonally upward beyond the limits of the apron before issuing. Width of apron, therefore, is important in fixing the practical factor of safety. Aprons should be submerged to form water cushions with depth equal to one-quarter, or preferably one-third the total hight of overfall, and width not less than 1½ times total hight of overfall, with a lip sloping upward and extending not less than 1½ times the length of sheet-piling in the main curtain wall, ending with, and supported on, a secondary line of sheet-piling having one-third to one-half the penetration of the curtain wall.

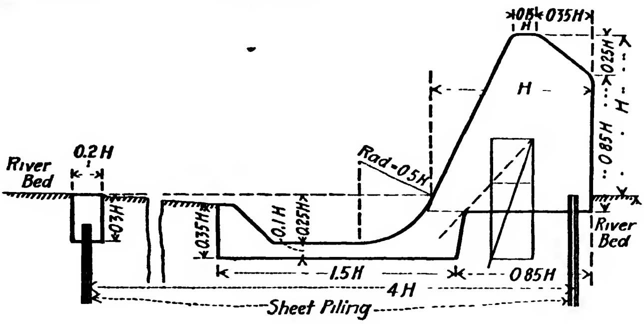


FIG. 69.

Considerable economy of concrete and greater safety against seepage is illustrated in Fig. 69, which assumes length of spillway such that maximum hight of overflow will not exceed $H \div 5$.

Downstream edge of water cushion should be at or slightly below the stream bed. At a point downstream, about twice the length of the main sheet-piling (or 4 or 5 times the head), there should be a second wall of tight sheet-piling with a penetration about one-half that of the main curtain-wall, parallel to the dam, to support a substantial concrete curb at the elevation of stream bed. Open space between water cushion and curb provides for escape of seepage under the curtain wall, while the curb prevents removal of sand, which, in this confined section, is subject to combined forces of upward and horizontal currents. For low dams there is a variety of cross-sections; for higher heads economy narrows choice to curved or stepped forms.

Under rules stated, a dam to raise water 50 ft., in a river having a bed of sand of unlimited depth, would require sheet-piling having minimum penetration of 80 ft. Instead, ample safety could be attained with steel sheet-piling

penetrating 50 ft. under the upstream side of the dam, and a secondary line with a penetration of 15 ft. under the extreme toe, which should be curved up to form a trough with a depth of 5 ft. below the river bed for a water cushion. About 200 or 250 ft. downstream construct a dam, 15 or 16 ft. high as described above, to back up water against the main dam for a water cushion. Seepage, which would probably find its way under the main dam in considerable quantities, would issue between the dams, where depth and length of water cushion prevents removal of sand.

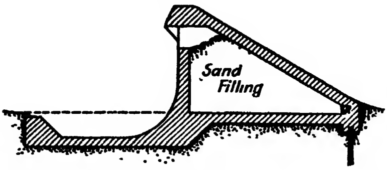


Fig. 70.—Method of obtaining weight for hollow dam.

In calculating the proportions of a dam to be founded on sand, there is involved hydrostatic pressure against the under side of the dam. Hydrostatic pressure against under side of any dam of the type illustrated in Fig. 69 is governed by the head of water on the downstream side and will vary between 25 and 40 per cent. of that which would result from the head of water on the upstream side.

Dams on alluvial foundations should be designed with the maximum width of base consistent with economy; therefore the gravity reinforced concrete dam is especially adapted to the requirements.

OVERFLOW DAMS: WASTE WEIRS

Forces Acting. Forces tending to destroy a waste weir or overflow dam are: (1) Water pressure due to (*a*) static head; (*b*) velocity of approach, considered as parabolic function varying from 0 at bed to 6 ft. per sec. at crest, and assumed to exert a horizontal pressure only; (*c*) buoyancy (under base), considered to have parabolic distribution from 0 at toe to pressure due to full flood head at heel. (2) Silt pressure. (3) Ice thrust. (4) Wave impact. (5) Impact of floating objects. (6) Vacuum effect under falling water. (7) Frictional effect of overfalling water in contact with downstream face. (8) Wind pressure. (9) Crushing of foundation.

Forces tending to hold the structure to position are: (1) Weight of masonry, acting by gravity alone. (2) Inertia of cross-section. (3) Silt and water pressure on battered heel. (4) Weight of overflowing water. (5) Pressure of backwater on toe. (6) Cohesion of masonry. (7) Mechanical bond at base. (E. R., Apr. 22, 1911.)

Vacuum Effects. According to G. S. Williams, experiments on a small model showed reduction of atmospheric pressure of one-sixth atmosphere (about 2.5 lb. per sq. in.) at the toe of an overflow dam, due to partial vacuum under the falling water. Tests were on a model 8 ft. high, with 2 ft. of water on crest. He estimated that this pressure might equal 12 lb. per sq. in. for a high dam. Ibid.

Air under Falling Water. Air is supplied to downstream face of an ogee type dam built by Tacoma Light and Water Co. across Green river, Washington, by 8-in. sewer pipes 20 ft. on centers extending from apron to air supply pipe, parallel to crest, leading to atmosphere at abutments. Maxi-

mum section of dam is 16.13 ft. high, with base width 24.10, designed for a factor of safety of 2.73 under 15-ft. flood head. Length of overflow, 150 ft. (Eng. Rec., Oct. 22, 1910). Like provisions in Pedlar river dam, p. 177.

Contour of Face. On Holyoke dam (see p. 177) with 4-ft. head on crest, the falling water is in contact with the face from crest to toe. This is true up to a head of about 5 ft., beyond which the surface of the sheet becomes so rough that it is not possible to investigate the contact.

Aprons. Horizontal aprons tangent to face curves at lowest points are better than "uplift" to give protection against erosion beyond the apron, as uplift causes water to strike another blow (A. F. Sickman).

Water Cushions for Overflow Dams.* Width of floor of cushion depends somewhat on section of dam, but need not exceed $8\sqrt{D_w}$ and should not be less than $6\sqrt{D_w}$. For works with a vertical drop and a horizontal apron on same level as toe of dam, this masonry apron should be formed of fairly regular blocks of stone or Portland cement concrete, the width varying as just given, and the thickness or depth of stone from $\frac{1}{3}(H + D_w)$ to $\frac{1}{4}(H + D_w)$.

$$D = C\sqrt{H^3\sqrt{D_w}}.$$

D = depth of cushion below top of subsidiary weir, see Fig. 71.
C = coefficient, dependent upon material used for floor of cushion; it varies between 0.75 for compact stone, and 1.25 for moderately hard brick;
H = hight of fall, from surface to surface;
D_w = maximum depth of water to pass over crest.

Greatest depth of hole formed by the waterfall in the new outlet to Mudduk Masur Tank (reservoir) is 24 ft. In ordinary stages of the river the general depth of the cushion is to the hight of fall as 3 to 4 where greatest action occurs, and 1 to 2 in other places.

An experimental fall on Bari Doab Canal (India) had a hight of 6.9 ft. and a depth of well of 9.0 ft. and 3.6 ft. on crest, which gives depth of well to hight of fall as 3 to 4. The water had no injurious effect on bottom of well.

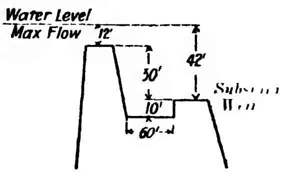


FIG. 71.—Periyar dam waste weir.

Periyar dam (India) waste weir. Head on subsidiary weir one and one-half times head on main weir. Fall from surface to surface varies from 24 to 30 ft. and depth of cushion from 10 to 28 ft. (See p. 186 for section of Periyar dam.)

Details of Flashboards. Las Vegas dam, New Mexico, was designed with water level at the top of the dam, since the reservoir is both fed and drained by conduits, the flow in which can be regulated. To allow for waves, Metcalf and Eddy provided flashboards 2 ft. high, with pins designed to bend under 2-ft. head on the dam crest, in case of a cloudburst, or the failure of the diversion conduit Since the dam is curved, failure of the pins might cause arch action in flashboards with butt joints, preventing removal until their failure under a higher head. To avoid this, every 50 ft., pins are placed in pairs, which will

* U S Geol Survey, 12th An Report, 1890-91, Part II, p 535 (Irrigation in India, by H. M. Wilson)

not fail with adjacent pins. The flashboards are provided with lap joints at these points. These double pins remaining steadfast, buckling of the flashboards results, and the water quickly flows off (E. R., Oct. 8, 1910, p. 404).

The Sudbury dam (Boston Waterworks) has flashboards 10 ft. long on the average. They are in 2 sets, the lower ones 8 in. high; the upper ones, 4 in. The boards are 3 in. thick, tapering at the ends.

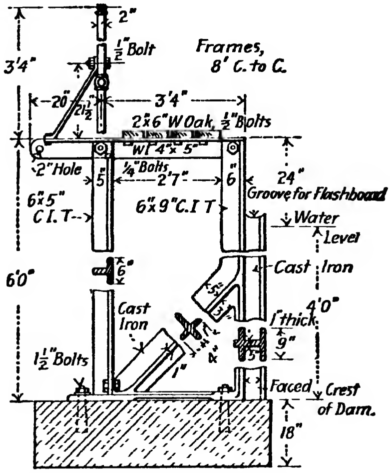


FIG. 72.—Flashboard standards, Muscoot dam, Croton reservoir, N. Y.

Flashboard Pins. Formula for Size and Spacing. (R. Muller, E. R., Aug. 22, 1908, p. 208.)

$$x = \frac{1,259,340d^3}{S\,lh^2} + \frac{2}{3}h.$$

x = hight of water, in feet, above dam crest when pins begin to bend.
h = hight of flashboards, feet.
l = spacing of pins, feet.
d = diameter of pins, inches.

Ultimate strength of pins, S, 69,500 lb. per sq. in. for Wayne iron, used by the Essex Co., Lawrence, Mass.

Table 45. Excessive Floods vs. Spillway Capacity

Stream	Station or country	Date	Watershed sq mi.	Equivalent run-off, in per 24 hrs	Flood flow cfs., per sq mi	Annual rainfall, inches
Catawba	Rockhill, S C	May, 1901	2,987	1 9	50	----
Catawba	Not stated	May, 1901	1,535	2 3	62	---
Cane Cr	Bakersville, N C	May, 1901	22	50 0	1,341	---
Elk Horn	Keystone, W Va *	June, 1901	44	51 0	1,363	----
Santa Catarina	Monterey, Mex	Aug, 1909	544	22 0	590	25
Estanzuela	Monterey, Mex	Aug, 1909	3 5	31 0	825	25
Tansa	India	Not known	52 5	25 0	667	101
Khrishna	India	Not known	345	13 0	343	258
Irawaddy	India	Not known	149,810	0 5	13	368
Ganges	India	Not known	367,970	0 2	5	27
Ohio	Cairo, Ill	Not known	214,000	0.1	3 3	55
Delaware	Lambertville, N J.	Not known	6,820	1 3	36	45
Colorado	Austin, Tex	Not known	37,000	0 1	3 3	24 5
Mississippi	St Louis	Not known	1,226,000	0 03	0 8	----
Nile	Assuan, Egypt	Not known	1,300,000	0 01	0 35	----

* Average flood flow for 14 hrs. = 278 cfs. per sq. mi.

Spillway Capacity (See also p. 98). In E. N., Nov. 2, 1911, Conway gives flood data (Table 45) showing impossibility of providing spillway capacity to take the greatest recorded floods. See also E. N., Sep. 23, 1909.

The Automatic crest, patented by G. F. Stickney, is placed on top of a dam to increase its hight, but arranged to drop when the water flowing over the crest exceeds a certain depth. The crest rises while the water is flowing over the dam, whenever the water surface falls to the proper stage. (See Fig. 73.) The automatic crest may be made any hight desired, is permanently attached so that it cannot float away, and it is raised and lowered solely by water pressure. It consists of two leaves, connected at an angle to each other, and hinged to a masonry base. The water above the dam has access to

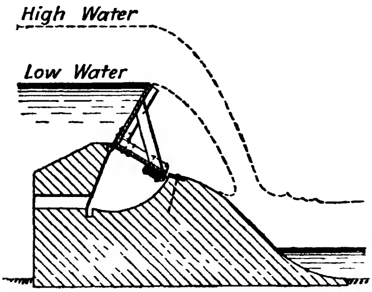


FIG. 73.—Automatic crest raised.

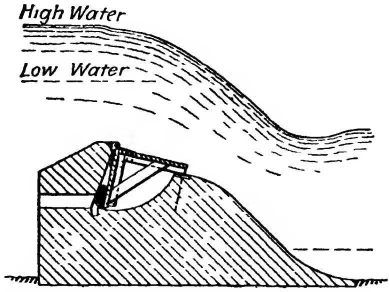


FIG. 74.—Automatic crest lowered.

the two leaves at all times. The water pressures against the leaves act in opposition to each other; the pressure against the lower leaf tending to raise the crest and that against the upper leaf tending to lower it. Operation is based on the principle that the center of pressure against any vertical or inclined surface rises as the water surface rises. The leaves are so proportioned that, so long as the water surface is below a certain level, the raising force preponderates and the crest will stand erect, and after this level is exceeded the lowering force becomes greater, whereupon the crest falls. When the crest is erect the leaves abut against contact strips, making tight joints all around. E. N., Feb. 15, 1912.

CONSTRUCTION

Cut-off Walls are used as an important aid to the prevention of flow between a dam and its foundation, and through porous or seamy foundation materials beneath the dam. A relatively narrow trench (5 to 20 ft.) is excavated to satisfactorily water-tight materials or to such depth as to offer great resistance to the water's passing it. Grouting is frequently done below the bottom of this trench, as described on p. 164. Commonly the cut-off trench is filled with very dense masonry, built against the sides of the excavation and carefully bonded to the dam. Cut-off walls are usually under the heel of the dam, but may follow a very irregular line to meet local conditions; they are also sometimes extended beyond the ends of a dam to reduce seepage around the dam. The probability of seepage around the ends of masonry dams was met in Derwent Valley Waterworks, England, by carrying a wing trench for about

half a mile along each side of the reservoir. The site of the reservoir was underlaid by strata of sandstone and shale. These layers were in many cases broken, and this condition was found to extend far into the hills back of the dam. (E. R., Aug. 12, 1911.)

Openings through dams near the base for temporary tunnels for the stream or roads or permanent large pipes cause undesirable concentrations of pressures, where pressures are naturally of maximum value. These openings should be carefully looked after in design, construction and maintenance.

Water Control during Construction of Dam. To dispose of the stream or other water occupying a dam site much expense and ingenuity are often demanded. A stream may be diverted through tunnel, flume, pipe or temporary channel, or may be carried across the foundation pit in pipe or flume. Quantity of water, character of stream and nature of site will dictate the method. Cofferdams are usually required in connection with carrying the stream across the pit, and diversion dams for most schemes of diversion. Pumping is always necessary to care for ground water, and leakage past the temporary dams and from the pipe or flume. Sometimes a succession of cofferdams is used, each unwatering a portion of the site and forcing the stream into the remaining portion of its channel. Naturally seasons of low flow are chosen for such operations. In the masonry first built special devices are generally provided for making a quick closure of the passage or passages finaly left for the stream. Simple, strong iron or wooden sluice or flap gates are often used for the first step and are then quickly backed with masonry, the remainder of the filling being built at convenience. Provision should be made for grouting around this plug to fill shrinkage spaces; plug should be bonded to sides of passage.

Cross River Dam. Two temporary dams were built, above and below the dam site, and paralleling the proposed dam. These were connected by 560 ft. of wooden flume, 16 ft. wide and 6.5 ft. deep, and two 5-ft. steel pipes, extending across the dam excavation, supported on steel bents, and enveloped by the dam as it rose. When the time came to fill the pipes with masonry, a wooden bulkhead was built across the flume just above the pipes, and rubble, rather than concrete, was placed in successive walls, about 2 ft. thick. At the top, spalls were rammed into fairly dry mortar to close the pipe as tightly as possible; grouting was done through 1½-in. wrought-iron pipes, placed 3 in. below the top. Fifty-eight bags of cement were used for grout in one pipe, forty-eight in the other.

Earth Foundations. It is not customary to build high masonry dams on earth foundations, because the pressure at the toe is so great that the earth would not offer sufficient resistance, but at the end of a dam enveloped by earth cones, there will be no excessive pressure at the toe; moreover, the weight of the embankment over the earth would tend strongly to prevent the earth from squeezing out. Under such circumstances, it is proper to build a portion of a masonry dam on very compact earth (hardpan or its equivalent) under competent engineering supervision.

Testing Porosity of Dam Foundations. At the Keokuk dam across the Mississippi river, foundations were tested by using compressed air exerting

four times the pressure of the anticipated head. Test holes, 4-in. in diam. and 30 ft. deep, were put down 36 ft. apart on the axis of the dam. Query: May not some air be objectionably pocketed in the rock so as to interfere with subsequent grouting, without being discovered? Instead, why not pump water, noting closely the quantity and pressure, and changes in both?

Laying Masonry.* For economy, safety and convenience, with few exceptions, 8 tons should be the maximum weight of stones or blocks used; up to 16 tons may be handled with unusually large equipment. If a good quarry is available near the dam site, coursed or random ashlar masonry can be furnished and built into the dam almost as cheaply as concrete blocks; a long haul might cast the balance on the side of the blocks. For massive work, involving heavy moldings or deep coves, concrete blocks would effect economy. It is cheaper to do cutting when the concrete is green, but preferable to wait until it has hardened.

Face joints should be horizontal if the batter is steep, but may be normal to the face if the slope of the dam is flatter; however the contractor claimed that the latter proved expensive at Wachusett dam. Joints should be filled full to the face when the stones are laid, and properly finished, preferably concave. Raking out joints and repointing at a later date does not give such good results as the pointing mortar frequently comes out. No case is known of stones having spalled when joints were filled to the face when the masonry was laid. Retaining walls in Philadelphia, built by the same contractor under two engineers, one of whom repointed, and the other finished joints as laid, illustrate this point. Within 1 yr. after completion, the portion repointed had suffered from frost about 25 per cent., due to mortar being forced out; at last reports, the other portion was in excellent condition.

On the upstream face of Wachusett dam, in addition to ordinary headers, (every third stone in every second course), a continuous course of headers was laid every 20 ft. to support the stones above; under these courses the joints were left open until the dam was built to full hight and the rubble interior had had a chance to compress. Headers were used in the downstream face with the same frequency as in the upstream, except that there were no continuous courses. Face joints were raked out and not pointed until the dam was nearly completed.

Concrete Blocks for Facing Dams. Use a mix of $1:2\frac{3}{4}:5\frac{1}{4}$ to $1:3:5$. The best finish is obtained by casting the blocks with exposed faces down and against a steel plate in the forms. For Cross River dam, concrete weighed 152 lb. per cu. ft.; the largest blocks contained 87.1 cu. ft., and weighed 6.62 tons. Three months seasoning should be allowed; this fact is demonstrated by the high percentage of breakage after shorter seasoning. 4.7 per cent of the blocks were rejected. The blocks constituted 11 per cent. of the total mass.

Similar blocks were used on Croton Falls, Olive Bridge, Kensico and other important dams, for a large part on the whole of one or both faces. Rejections on the later dams were a very small percentage. The course hight was commonly 2.5 ft. Blocks were used also for one face of each expansion joint in some dams. One or two courses were laid in advance of the cyclopean or other hearting masonry and served as forms for it. Edges of exposed faces of blocks

*Only a few special notes are given here as the general subject has been discussed at length in other books.

should be rounded to a radius of about 1 in. In Olive Bridge and Kensico dams very lean, porous concrete blocks, with 16-in. diam. holes were used for vertical drainage wells near the upstream face.

GROUTING FOUNDATIONS

Cross River Dam. Drilling and blasting were allowed until seemingly hard rock was reached, when barring-and-wedging was resorted to for removing loose portions. At times this would disclose rotten rock below, calling for more blasting. The final bottom was washed clean, and sounded with a heavy hammer in the presence of an inspector. Mortar was spread over this wet surface just before masonry was placed. Wells of rubble masonry were built around springs. Vents were left for grouting, all springs being free to rise to their hydraulic grade line. A Douglas No. 2 bilge pump was used for grouting.

New Croton Dam. Seams, erosions and caves in rock were thoroughly excavated, then cleaned with a stream of water under high pressure from a nozzle. Larger spaces generally were packed with rubble masonry, smaller spaces poured full of grout of Portland cement and fine sharp sand, 2 to 1, wherever possible. Where pouring was not possible, grout, mixed 1 to 1, was pumped. Connections to some seams were made by drill holes; a few caves were entered by small shafts and tunnels. Water encountered under pressure was forced into pipes sealed into the rock and carried up above the level to which water rose, before grouting. Some of these pipes were 10 in. diam., but most were 2 in. Preliminary to grouting, pipes, etc., were thoroughly flushed with clean water under high pressure. One spring yielded 1.5 mgd. For one spring 2-in. pipes were carried up 90 ft. and, it proving impracticable to grout through so great a depth of water, plastic blue clay, well kneaded with water, was rammed in with a 2000-lb. pile driver, with maximum drop of 4 ft., to a total of 22 cu. yd. (For further details, see "Foundations of New Croton Dam" by C. S. Gowen, T. A. S. C. E., Vol. 43, 1900, p. 469.)

The Clackamas river dam of the Portland (Ore.) Ry., Light & Power Co., is founded on volcanic andesite and basalt, heavily fissured. A double line of holes was drilled under the heel of the dam to average depth of 50 ft. Canniff pneumatic grout mixer and injector was used, with air pressure from 50 to 200 lb. per sq. in., depending on the tightness of the hole; high pressure caused blow-outs at surface. Holes were 2⅜ to 8 in. diam.; 3-in. wrought-iron pipe casings, with threaded tops for attaching hose, were put down for upper 4 to 6 ft., and grouted in. Grout contained 2 to 15 parts water to 1 cement. A flexible copper hose connected the grout machine to the pipe. Introducing the hose into the pipe for grouting low parts was tried, but did not prove worth extra work. Holes were tested under a head exceeding the normal reservoir head; in some cases, communication was found between holes 70 ft. apart. Drilling and grouting 555 holes, aggregating 34,038 ft., cost $1.55 per ft. (H. V. Schreiber, E. R., Mar. 23, 1912.)

At Olive Bridge dam of Catskill water supply for New York, 29 grout holes, 3 in. diam., 5 ft. apart, and 13 to 20.8 ft. deep, were drilled in the cut-off trench down to a seam indicated by previous borings. When this seam was

tapped, water rose under pressure to the floor of the cut-off trench. Two-inch grout pipes were chipped with a chisel at the lower end, wrapped with jute for about 8 in., and driven 3 ft. into the drill holes, the jute on the lower end fitting the hole tightly. The annular space outside of the pipe was grouted, and filled with more jute wound around the pipe and calked so firmly that all flow of water stopped. Pipes were finally sealed to rock with neat Portland cement. As the cut-off trench was filled, the 2-in. grout pipes were carried up with the concreting to a proper hight for grouting. A Cockburn-Barrow grouting machine of about 4 cu. ft. capacity was used. This machine consists of a cylinder, 1.75 ft. diam. and 3 ft. long, fitted with paddles to churn the grout. Materials are introduced through a hole in the top. Pipes admit air under pressure which forces grout through a 2-in. hose, connecting by a tee to the grout pipe. A throttle valve in the tee permits the blowing off of the hose without disconnecting it from the pipe. Air pipes are controlled by screw valves; outlets to hose, by throttle valve. A pressure gage is attached to the air pipe. A compressed air engine attached to the machine keeps the stirring paddles in motion. Prior to grouting, water stood 20 ft. below the tops of the pipes. Under 25 lb. pressure, water was immediately forced from two adjacent holes, spurting 6 ft. above the pipe-mouths. Air pressure was then connected to the grouting machine and the holes grouted. The volume of pipe and drill hole was about 1.4 cu. ft.; 3.2 cu. ft. of grout were required to fill them.

State Control of Dams

(E. R. Jan. 6, 1912)

State	Supervisory official	State	Supervisory official
Colorado	State engineer	New Mexico	Irrigation engineer
Connecticut	State board, headed by engineer of Railroad Commission	New York	Inspector of dams and wharves
Florida	County commissioners	Oklahoma	State engineer
Georgia	Committee of freeholders	Oregon	State engineer
Idaho	State engineer	Pennsylvania	State water supply commission
Indiana	County commissioners	Rhode Island	Commissioner of dams and reservoirs, appointed every 3 yrs by Governor
Kansas	County court, laws define methods of construction	South Carolina	Committee of freeholders
Maine	Civil engineer, appointed by Governor annually	South Dakota	State engineer
Massachusetts	County commissioners	Tennessee	County court
Michigan	County board of supervisors	Texas	County commissioners' court
Montana	Civil engineer commission appointed by county court	Utah	State engineer
Nebraska	State board of irrigation	Vermont	Commission of three engineers appointed for specific cases by justice of supreme court
Nevada	Local board of water commissioners		
New Jersey	Board of three engineers, appointed by governor for each case	Wyoming	State engineer

Typical Masonry Dams. In Tables 46 and 47, following, dams are first arranged in classes, overflow and non-overflow, and under the 2 classes, the arrangement is alphabetical. As in previous tables, use of........means that the condition does not exist, and use of — — — — — — —means lack of knowledge. "Hight above lowest foundation" is exclusive of cut-off trench. Sections of these dams are on pages 174 to 190, in almost the order of the table. "Max. pressure" is on plane normal to resultant, unless noted otherwise.

Table 46. Typical Masonry Dams (Overflow)

DIMENSIONS IN FT.

Name	Austin (Colorado river)‡	Betwa	Boonton	Butte City	Catawba river
Locality	Texas	India	New Jersey	Montana	So. Carolina
Year built	1891–92	1888–97	1900–5	1893–95	1903–04
Hight above stream-bed	64	------	60	------	34
Hight above lowest foundation	68	61 5	95	120(a)	55
Length on top (overflow section)	1091	3296	300	350(b)	585
Minimum thickness at or near top.	Rounded	15	Rounded	20	Rounded
Maximum thickness at or near bottom °	66	61 5	65	83	33
Radius of upstream face (if curved in plan).		Follows reef		350	
Flood depth for which designed	------	16 4	5	------	20
Character of foundation	Limestone	Rock ledge	Shale and sandstone	------	Granite
Kind of masonry	Rubble	Granite rubble	Cyclopean syenite	Cyclopean	Cyclopean rubble
Maximum pressure on heel (tons per sq ft)	------	------	------	------	------
Maximum pressure on toe (tons per sq. ft.)	------	------	------	------	------
Remarks	Failed, under 11 07 ft. on crest	Usual profile reversed so that 6″ overflow would clear face Has had 16 4 ft head on crest	------	(a) Built only 80 ft to date (b) 126′ to date	
References	E N, July 11, 1891	H M. Wilson 12th An. Report, U S G S, 1891	Wegmann* E R, Aug 8, 1903	E N, Sept 5, 1895; Dec 15, 1892	E. R, July 23, 1904

Table 46. Typical Masonry Dams (Overflow)—*Continued*

DIMENSIONS IN FT.

Name	Connecticut river	Dunnings (Scranton)	Escanaba	Folsom	Great Northern Power Co
Locality	Vernon, Vt	Pennsylvania	Michigan	California	Minnesota
Year built	1907–10	1887–88	1910	1886 91	1907
Hight above stream-bed	39	56	26	70	38
Hight above lowest foundation	70	67±	26	98	40
Length on top (overflow section)	600	152 6	461	180	365
Minimum thickness at or near top.	Rounded	10	Rounded	24	Rounded
Maximum thickness at or near bottom	33 (a)	56	23 3	87	42
Radius of upstream face (if curved in plan)			..		
Flood depth for which designed.	Sluices in dam	5	2	6	6
Character of foundation	Rock	Rock and clay	Limestone	Bed rock	Rock
Kind of masonry	Cyclopean concrete	Rubble masonry	Cyclopean concrete	Granite ashlar	Concrete
Maximum pressure on heel (tons per sq ft).	------	------	------	------	------
Maximum pressure on toe (tons per sq ft).	------	------	------	------	------
Remarks	Flashboard 4 ft high (a) 28′ below top	....	1-ft flashboards. 8″ pipes at 10′ intervals drain toe.	Movable shutter raises level 5 ft.	....
References	E R, Mar 27, 1909	Trans Am. Soc C. E, Vol 32, 1894	E R, May 15, 1909	† Schuyler, p 264	E. R, Sept 7, 1907

* Wegmann, "The Design and Construction of Dams." ‡Replaced in 1914. See E. R, June 3, 1915, showing later design

† Schuyler, "Reservoirs for Irrigation, Water-Power, and Domestic Water Supply, 1908

° "Max. thickness at or near bottom" for overflow dams, includes masonry apron, if any, built as part of the dam section.

Table 46. Typical Masonry Dams (Overflow).—*Continued*

DIMENSIONS IN FT.

Name	Hale's Bar	Holyoke (New)	Keokuk (Miss river)	Lynchburg (Pedlar river)	McCallsFerry (Susquehanna river)
Locality	Tennessee	Massachusetts	Iowa & Ill	Virginia	Pennsylvania
Year built	1910–13	1897	1911–13	1904–08	1904–10
Hight above stream-bed	62	30	32	52	53
Hight above lowest foundation.	76	38	37	67.5	60
Length on top (overflow section)	1200	1020	4278	150	2350
Minimum thickness at or near top.	Rounded	Rounded	6	9	Rounded
Maximum thickness at or near bottom	66	51 2	42	44	65
Radius of upstream face (if curved in plan)					
Flood depth for which designed.	------	4	11	6	17 5
Character of foundation	Limestone seamy in spots	------	Limestone	Shale	Hard gneiss
Kind of masonry....	Concrete	Rubble	Concrete	Concrete and granite	Cyclopean rubble
Maximum pressure on heel (tons per sq ft)	------	------	------	------	4.8
Maximum pressure on toe (tons per sq ft)	------	------	------	------	6 2
Remarks....... .	Caisson foundations		------	Air pipes prevent vacuum at face	.
References	E R, Feb. 15, 1913. E. N, Nov. 13, 1913	E N., 1897, Vol. 37, p. 292	E N, Sept 28, 1911	E R, May 12, 1906, July 23, 1904	E R, May 28, 1910

Table 46. Typical Masonry Dams (Overflow).—*Concluded*

DIMENSIONS IN FT.

Name	Muscoot	Scioto (Columbus)	Spier Falls	Turlock (LaGrange)	Vyrnwy
Locality	New York	Ohio	New York	California	England
Year built	1902	1904–5	1900–05	1891–94	1882–90
Hight above stream-bed	35	33	52	127 5	101
Hight above lowest foundation	70	45 5	70	127 5	162
Length on top (overflow section)	200	500	817	309	1172
Minimum thickness at or near top	5 5	Rounded	Rounded	Rounded	Rounded
Maximum thickness at or near bottom	31 5	64 3	65	92	120
Radius of upstream face (if curved in plan)		800		308 9	
Flood depth for which designed	------	22	------	16	1 5
Character of foundation	Rock	Limestone	Granite	Rock	Clay, slate, rock
Kind of masonry.	Rubble masonry	Concrete	Rubble, cyclopean	Blue trap rubble	Cyclopean rubble
Maximum pressure on heel (tons per sq ft)	------	------	------	------	8 7
Maximum pressure on toe (tons per sq ft).	------	------	------	------	6 36
Remarks	Downstream face stones carefully selected and bedded to withstand ice	Allowance for future addition 22' high	. .....	Has had 15 ft. head on crest	Foundation rock drained to prevent uplift
References	E R, Dec 20, 1902	T A S C E, Vol 67, 1910	E N, Jun 18, 1903	E N, 1894, Vol. 31, p 266	Proc Inst C E, Vol 126, 1895, p 29

Table 47. Typical Masonry Dams (Not Overflow)
DIMENSIONS IN FT.

Name	Ashokan (Olive Bridge)	Assuan	Barker	Barossa	Barren Jack
Locality	New York	Egypt	Colorado	S. Australia	N. S. W
Year built .	1907–11	1898–05	1909	1899–03	1908
Hight above stream-bed	150	82 (a)	145	95	224
Hight above lowest foundation (exclusive of cut-off trench)	220	131	172	113	240
Length on top (omitting spillway).	1000	6400*	625	472	784
Minimum thickness at or near top.	26 3	17.8	16	4 5	18
Maximum thickness at or near bottom.	190 2	85	124	45	160
Radius of upstream face (if curved in plan)	.			200	1200
Hight of top above full res level.	20	13 1	———	2	12
Character of foundation	Hamilton shale	Granite ledge	Granite. 10 ft cut-off wall 10 ft. deep	Shale	Granite
Kind of masonry......	Cyclopean	Red granite rubble	Cyclopean	Cyclopean	Cyclopean
Maximum pressure on heel (tons per sq ft)	14†	6 5†	———	———	16.8
Maximum pressure on toe (tons per sq ft)	8 2†	4 5†	———	———	15 5
Allowed ice thrust, tons per lin ft at full res level	23 5	None	———	———	———
Allowed for upward water pressure.	Full head at heel and 0 at toe, applied on $\frac{2}{3}$ area of joint	None	———	———	———
Remarks	Continuous drains; wave hight figured on gale of 40 mi. per hr.	180 sluices each 6 56 ft wide	Expansion joints 48 ft apart, begin 140 ft below top. Hot asphalt	Pressure assumed 15 tons, rail reinf in top 15 ft.	Factor of safety, 2 16
References	....	E. N., Sept. 30, 1909, Cassier's, Feb , 1903	E R , Oct 2, 1909	E R , Sept 2, 1905, E N , Apr 7, 1904	Engineer, Sept 23, 1910

* Including both pierced and solid dam. (a) Increased 16′ in 1911 to 98 ft
† Vertical pressures

Arrowrock dam, of U. S. Reclamation Service on Boisé river, Idaho, is highest in the world. Built 1912–15. Maximum hight above foundation, 351 ft.; above river level, 260 ft. Length on top, 1050 ft. Minimum thickness, 15.5 ft; maximun, 238 ft. Radius of center line of top, 661.74 ft. Parapets on each side of roadway extend 4 ft. above top of dam. There is a waste weir 400 ft. long, separate from the dam, with crest about 5 ft. below top of dam; but the dam is designed to permit a flow 2 or 3 ft. deep over its top in great floods. Cross section is of gravity type, with vertical upstream face, the portion below 100 ft. being offset 3 ft. upstream. Foundation is hard granite. The dam is of concrete cast against wooden forms; proportions: 1 sand cement, 2½ sand, 5½ gravel and 2¾ parts cobbles, with a richer mixture for 10 ft. from each face. Sand cement was made of 55 per cent. Portland cement and 45 per cent. pulverized granite, reground to pass 90 per cent. through a sieve having 200 meshes per inch. There is a 20-ft. inspection gallery, 25 ft. from the upstream face, with its bottom 235 ft. below top of dam. Above this gallery

are 6″ weeper drains, 10 ft. center to center, sloping uniformly to within about 6 ft. of upstream face 25 ft. below top of dam; below the gallery are 12″ drains, 10 ft. center to center, extending vertically into the rock beneath the dam. There are vertical contraction joints, normal to the faces of the dam, 150 ft. apart.

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name	Bear Valley*	Beetaloo	Boonton	Boyd's Corners	Burraga
Locality	California	S Australia	New Jersey	New York	Australia
Year built	1884	1888–90	1900–05	1866–72	1893
Hight above stream-bed	------	110	105	58 0	39
Hight above lowest foundation (exclusive of cut-off trench).	64	118	114	78 0	41.0
Length on top (omitting spillway).	300	367	1850	670	425.6
Minimum thickness at or near top.	3 17	14	17	8 6	2.0
Maximum thickness at or near bottom.	8 4 at 48 below top, 20 max	110	77	57	25 3
Radius of upstream face (if curved in plan)	335	1428			539 8
Hight of top above full res level	4	5	5 0	3	2 5
Character of foundation	Granite	Shale and quartzite	Shale and sandstone	Rock	Slate
Kind of masonry.....	Granite rubble	Concrete	Cyclopean of syenite	Concrete hearting, cut-stone facing	Cyclopean
Maximum pressure on heel (tons per sq ft.).	------	7 0	9	------	------
Maximum pressure on toe (tons per sq ft)	------	4 6	8	------	------
Allowed ice thrust, tons per lin ft at full res level	------	------	None	------	------
Allowed for upward water pressure.	------	------	None	------	------
Remarks:	Pressure limited to 40 tons Max arch stress = 59 tons per sq. ft	Pressure of 5 6 tons 82 8 ft below top Temperature cracks, ⅛″ wide, were grouted			Pressure limited to 10 tons, spillway in center; 3 rails laid near top to care for expansion
References . . .	E N, June 23, 1888	Proc. Inst C. E., 1892, Vol. 113, p 151	E. R, 1903, Vol. 48, p 153	Wegmann	Proc Inst C E., Vol. 152, 1903

* This is the old dam, which was the first of thin-arch dams. In 1911 a dam of novel type of reinforced concrete multiple arch construction was completed a short distance downstream, inundating the old dam. The new dam is 363 ft. long, of 92 ft. max. hight, consisting of 11 buttresses 32 ft. apart and 10 arches. The buttresses are braced one from another by struts; they are 12 in. thick at top and increase uniformly to bottom, the highest being 26 in. thick at the base. Each end arch contains spillway openings, the combined capacity being 1220 c. f. s. Area of watershed is 56 sq. mi. See illustration on p. 171. Ref. E. N., Dec. 25, 1913 and Oct. 28, 1915.

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name	Burrator	Cataract	Chartrain	Chemnitz (Einsiedel)	Cross River
Locality ...	England	Australia	France	Germany	New York
Year built	1893–96	1902–08	1888–92	1894	1905–07
Hight above stream-bed	89	157	163	65	155
Hight above lowest foundation (exclusive of cut-off trench).	145	192	180	92	170
Length on top (omitting spillway).	361	811	------	590	772
Minimum thickness at or near top.	18	16.5	13.12	13	23
Maximum thickness at or near bottom	63†	158	159 9	65.5	116
Radius of upstream face (if curved in plan)		....	1319	1316.5	
Hight of top above full res level	12±	6.6	1 5	1 64	10
Character of foundation...	Granite bed	Sandstone	Rock	Slate	Gneiss
Kind of masonry..........	Granite blocks in concrete	Cyclopean	Granite rubble	Rubble	Cyclopean
Maximum pressure on heel (tons per sq ft)	------	*	Limited to 11 3	------	10 0
Maximum pressure on toe (tons per sq ft)	------	*		------	12 5
Allowed ice thrust, tons per lin ft at full res level	------	None	------	------	12
Allowed for upward water pressure	------	None	------	------	See Ashokan
Remarks . . .		Drains			
References .	Proc Inst C E, Vol 146, 1901	E N, Dec 6, 1906, June 23, 1910, E R, June 6, 1908	Wegmann	E R, July 28, 1894	E R, Sept. 14, 1907, June 16, 1906

*Max allowable pres on heel and toe taken small (9.5 tons) on account of moisture-laden sandstone used † 77′ below water level

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name .. .	Croton Falls	Furens	Gileppe	Granite Springs	Hemet
Locality	New York	France	Belgium	Wyoming	California
Year built .	1906–11	1862–66	1869–75	1903–04	1890–95
Hight above stream-bed	113	170 6	151	86 ±	122 5
Hight above lowest foundation (exclusive of cut-off trench)	167	183 7	154	96	135 5
Length on top (omitting spillway)	1095	330	771	410	260
Minimum thickness at or near top	23	9.9	49 2	10	10
Maximum thickness at or near bottom	127.7	161	215 9	56	100
Radius of upstream face (if curved in plan)	.. ....	841.4	1665	300	225 4
Hight of top above full res level.	12	6.56	6 6	3	1 0
Character of foundation. ..	Granite	Mica schist	Rock	Gabbro	Bedrock
Kind of masonry	Cyclopean	Rubble	Rubble	Uncoursed rubble	Granite rubble
Maximum pressure on hee (tons per sq t)	10 0	Limited to 6 6	------	------	------
Maximum pressure on toe (tons per sq ft).	12 5		6 14	------	------
Allowed ice thrust, tons per lin. ft. at full res evel	15	------	------	------	------
Allowed for upward water pressure	See Ashokan	------	Sp gr of stone reduced from 2 3 to 1 3	------	------
Remarks..... ..		..... .	.		Designed for ultimate hight of 160 ft
References	Proc. Mun. Eng., 1910, E. R., Mar 28, 1908	Wegmann	Wegmann E. N, Dec. 25, 1886	E R., 1905, June 24, p 698; E. N, 1905, June 29, p. 671	18th An. Rept., U S. G S, 1897

For Crystal Springs dam, see San Mateo, p. 172.
For Croton or Cornell dam, see New Croton, p. 171.

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name ..	Hijar	Kensico	Lake Cheesman	New Croton	Pathfinder
Locality	Spain	New York	Colorado	New York	Wyoming
Year built	1880	1910–16	1900–04	1892–1907	1906–09
Hight above stream-bed	——	170	214	163	204
Hight above lowest foundation (exclusive of cut-off trench)	141.1	300	236	297	218
Length on top (omitting spillway).	236.2	1630†	710	1168	425
Minimum thickness at or near top	16.4	27.75	18	18	10
Maximum thickness at or near bottom	117	227.7	176	206	94
Radius of upstream face (if curved in plan).	218		400		157
Hight of top above full res level	4.9	15	3	10	10
Character of foundation .	Rock	Schist, limestone and gneiss	Granite	Seamy rock	Red granite
Kind of masonry........	——	Cyclopean	Granite rubble	Rubble	Cyclopean
Maximum pressure on heel (tons per sq ft).	6.0	15*	(a) 12.8 (b) 13.4*	†	Designed as horizontal arch including temp stresses Max = 13 tons
Maximum pressure on toe (tons per sq ft).	5.1	13*	17.7 13.9	†	
Allowed ice thrust, tons per lin ft at full res level	——	23.5	None	None	——
Allowed for upward water pressure.	——	Full head at heel and 0 at toe, applied on ⅔ area of joint	None	None	——
Remarks. ..		Continuous drains Stones weighed 1 to 16 tons	(a) Computed as gravity section, (b) arch and cantilever	Spillway 1000 ft long	.
References	Wegmann	†Exclusive of pavilions	T A S C E, Vol 53, 1904 p 89	E R, Aug 16, 1902, E N, Oct 4, 1906	E N, Oct 29, 1908, E R, Nov 7, 1908

* Vertical. † Profile is based on Quaker Bridge dam, which had max. heel pressure of 16.6 tons; max. toe pressure of 15.4 tons (Wegmann)

Old and new Bear Valley dams.
(*Eng. News, Oct.* 28, 1915.)

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name	Periyar	Roosevelt	San Mateo*	Shoshone	Sodom
Locality	India	Arizona	California	Wyoming	New York
Year built	1888–96	1905–11	1887–88	1905–10	1888–93
Hight above stream-bed	155	262	146	243	78
Hight above lowest foundation (exclusive of cut-off trench).	178	284	162	328	98
Length on top (omitting spillway).	1231	1080	680	175	500
Minimum thickness at or near top.	12	16	25	10	12
Maximum thickness at or near bottom	136	170	176	108	53
Radius of upstream face (if curved in plan)	.. .	418	645	155	......
Hight of top above full res level	5	20	8	10	10
Character of foundation.	Bedrock (syenite)	Bedrock, sandstone	Fissured sandstone	Solid granite	Rock
Kind of masonry.	Concrete	Cyclopean	Concrete blocks	Concrete	Coursed rubble
Maximum pressure on heel (tons per sq ft).	>9	22†	------	Designed as horizontal arch including temp stresses	------
Maximum pressure on toe (tons per sq ft)	>9	23†	------		------
Allowed ice thrust, tons per lin ft at full res level	None	------	------	------	None
Allowed for upward water pressure	------	------	Whole head	------	None
Remarks	. .	. .	See p 125	.....	....
References	12th An Rept, U S G. S, 1891	E N, 1905, Vol 53, p 34, Sept 10, 1908, p 265	U S G S, 18th Report, Part IV, 1897	E N, Dec 9, 1909, E R, July 23, 1910	T A S.C E, Mar., 1893

Table 47. Typical Masonry Dams (Not Overflow).—*Continued*

DIMENSIONS IN FT.

Name	Spier Falls	Sweetwater	Sweetwater (Extension)	Thirlmere	Titicus
Locality	New York	California	California	England	New York
Year built	1900–05	1887–88	1910–11	1886–93	1890–95
Hight above stream-bed	90	75	110	58 5	109
Hight above lowest foundation (exclusive of cut-off trench)	154	94	129	112	135
Length on top (omitting spillway)	552	380	380	------	534
Minimum thickness at or near top	17	12	12	18.5	20 7
Maximum thickness at or near bottom	113	46	76	51.7	81 4
Radius of upstream face (if curved in plan).	.. ..	222	222	118.5	....
Hight of top above full res level	10	5.5	5	6.0	5
Character of foundation.	Granite	Porphyry	Porphyry	Ledge rock	Rock
Kind of masonry	Rubble, cyclopean granite	Rubble	Cyclopean	Cyclopean concrete	Rubble, rough coursed
Maximum pressure on heel (tons per sq ft).	6 6	5 8	8.14(*a*)	------	------
Maximum pressure on toe (tons per sq ft).	6.6	15.8	9.45(*a*)	------	------
Allowed ice thrust, tons per lin ft at full res level	------	None	None	------	------
Allowed for upward water pressure.	------	------	------	------	------
Remarks	Silt in water for 60. ft at mid-depth, sp. gr. = 1 6	In 1895, dam was over-topped with 22 in water for 40 hrs. without injury	Addition bonded to face of old dam (*a*) Vertical	Plan, reverse curve, to follow ledge	. ...
References	E N., 1903, Vol 49, p 553	T A. S. C E, Vol. 19, 1888, p. 201	E. N., Mar. 30, 1911, p. 371	Proc. Inst. C E., Vol. 126, 1895, p 5	E. R, 1895, Vol. 32, p. 58

* Also known as Crystal Springs dam. † "Construction of Masonry Dams", C. W Smith, 1915.

Table 47. Typical Masonry Dams (Not Overflow).—*Concluded*

DIMENSIONS IN FT.

Name.	Upper Otay†	Urft	Villar	Wachusett	Wigwam (Waterbury)	Zola
Locality	California	Germany	Spain	Massachusetts	Connecticut	France
Year built . . .	1900	1901–04	1870–78	1900–06	1893–94	1843
Hight above stream-bed	———	175	162	146	77	119 7
Hight above lowest foundation (exclusive of cut-off trench).	84	190	170 33	207	91	123 2
Length on top (omitting spillway).	350	741	349	850	600	205
Minimum thickness at or near top.	4	18	14 75	22 5	12	19
Maximum thickness at or near bottom	14	165 5	154 6	187	62 08	41 8
Radius of upstream face (if curved in plan)	361	656	447	. . . . . .	600	158
Hight of top above full res level	———	3	8 25	20	7	———
Character of foundation	Porphyry	Granite schist	Rock	Granite and schist	Gneiss and sandstone	———
Kind of masonry	Masonry reinforced by plates and cables	Trap masonry	Rubble	Granite rubble	Rubble masonry	Rubble
Maximum pressure on heel (tons per sq. ft).	———	———	———	12 6*	———	8 12, 62′ above base
Maximum pressure on toe (tons per sq ft)	———	———	9 60	10 7*	———	
Allowed ice thrust, tons per lin ft at full res level	———	———	———	23 5	———	———
Allowed for upward water pressure.	———	None	———	Full head at heel, 0 at toe applied on ⅔ area of joint	———	———
Remarks	.	Continuous drains		. . . . . .	Designed for 90 ft hight	Under full head pressure line falls 11 5 ft outside of base
References	Wegmann	E N, July 16, 1903, Engineering, Nov, 1907	Proc Inst C E, Vol 71, p 379, 1883	E R, Sept 8, 1900, E R, Oct 6, 1906	E N, May 7, 1903	Wegmann

* Vertical † 3′ of water passed over crest, Jan. 1916, without damage.

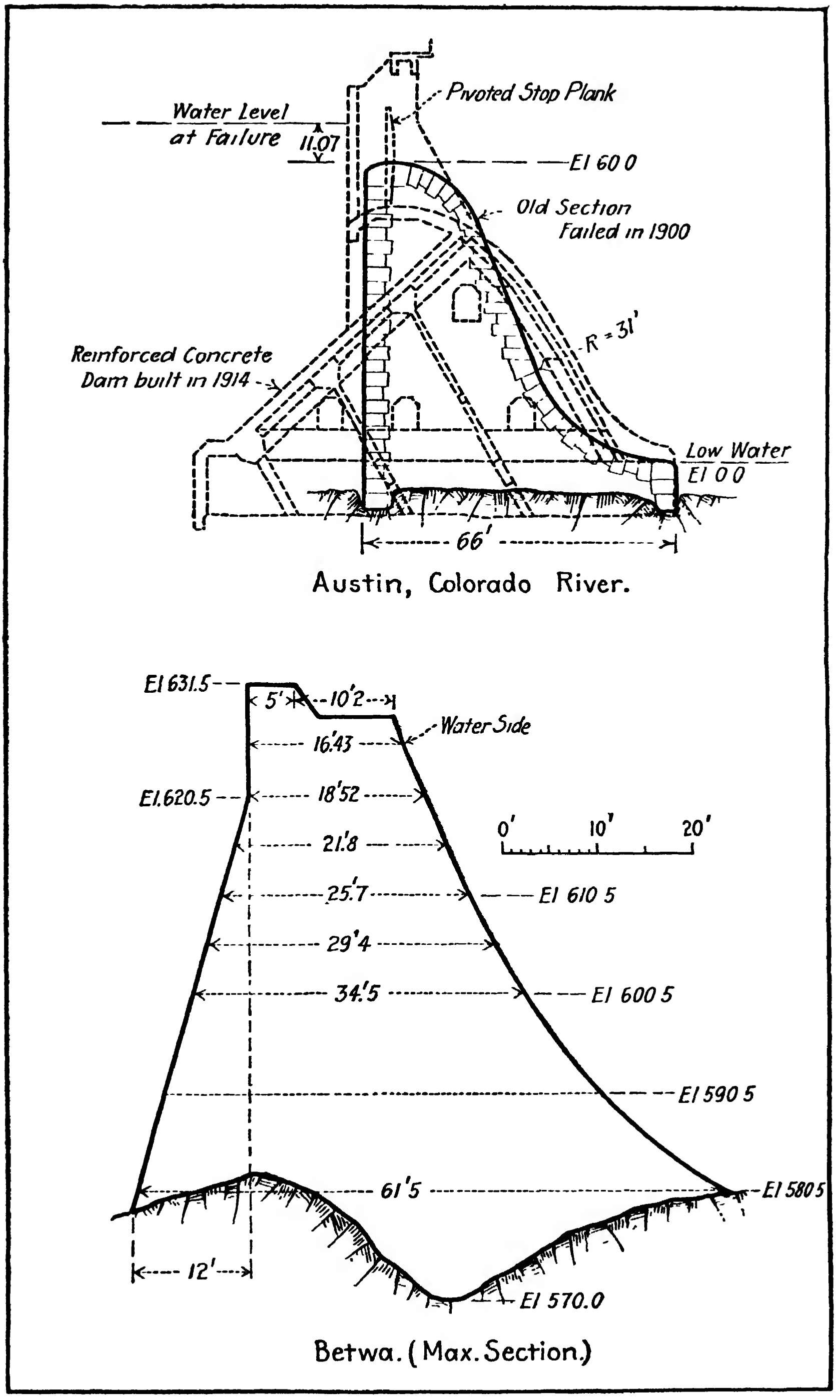


Fig. 75. Overflow Dams.

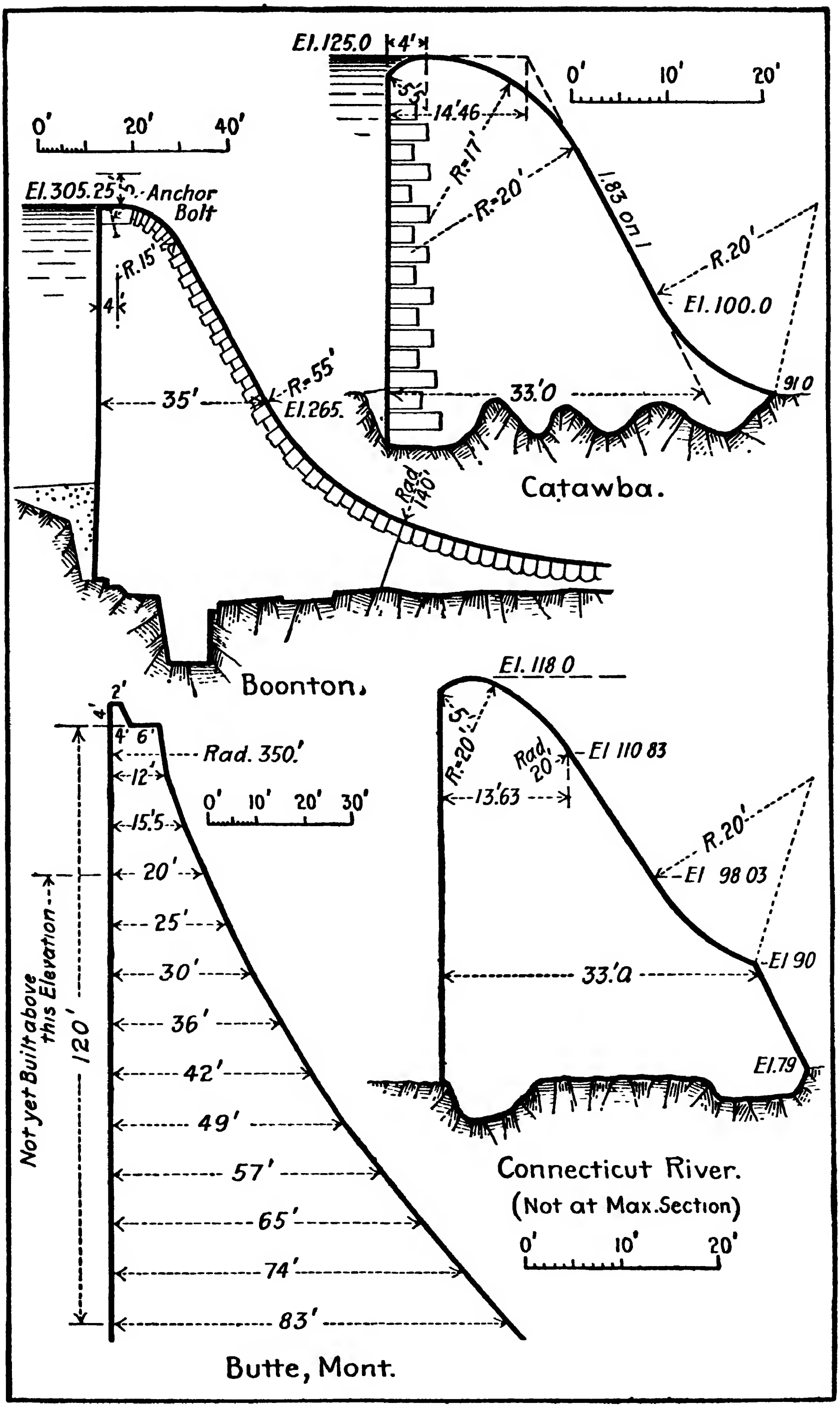


Fig. 76.—Overflow dams.

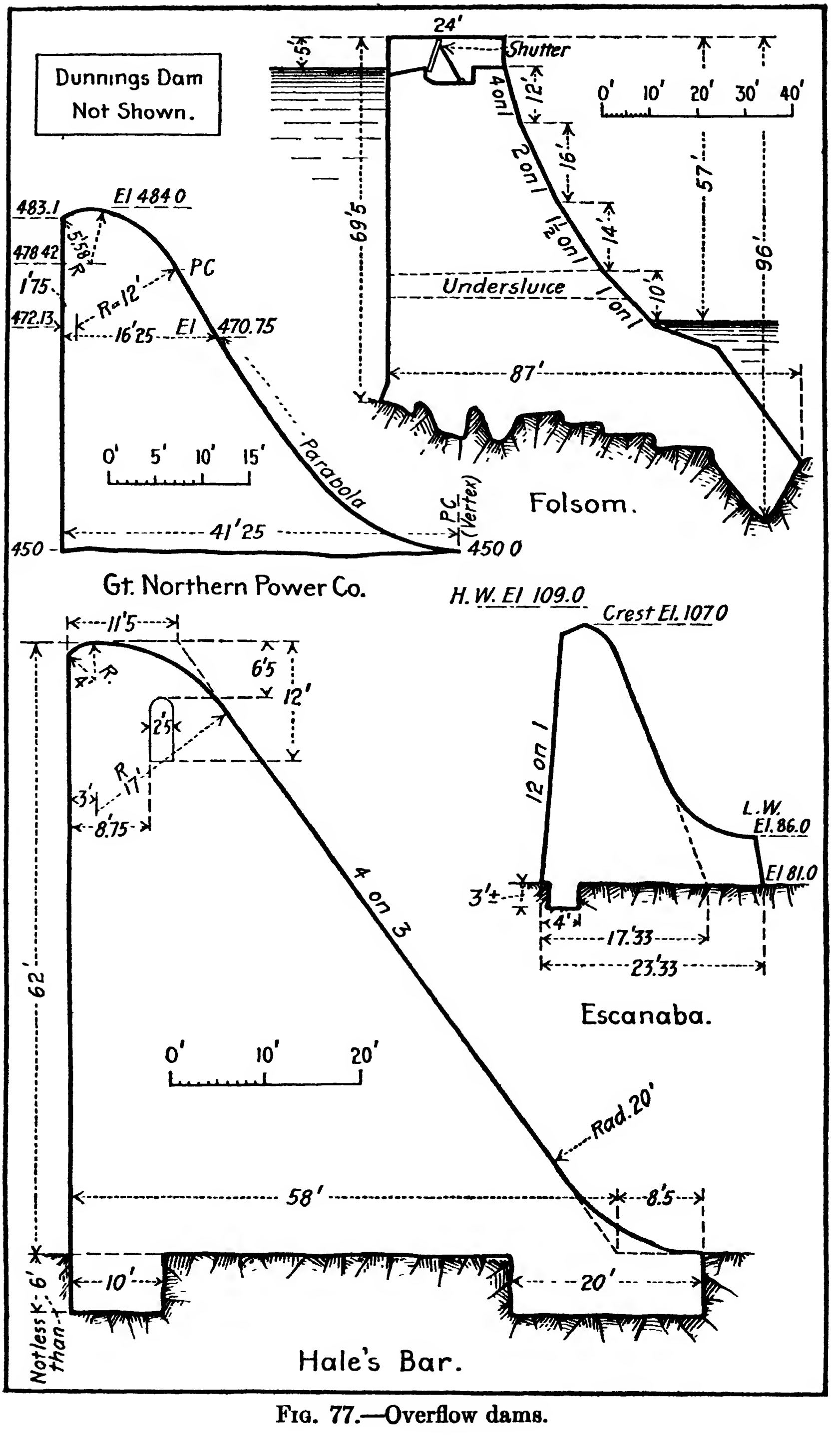


Fig. 77.—Overflow dams.

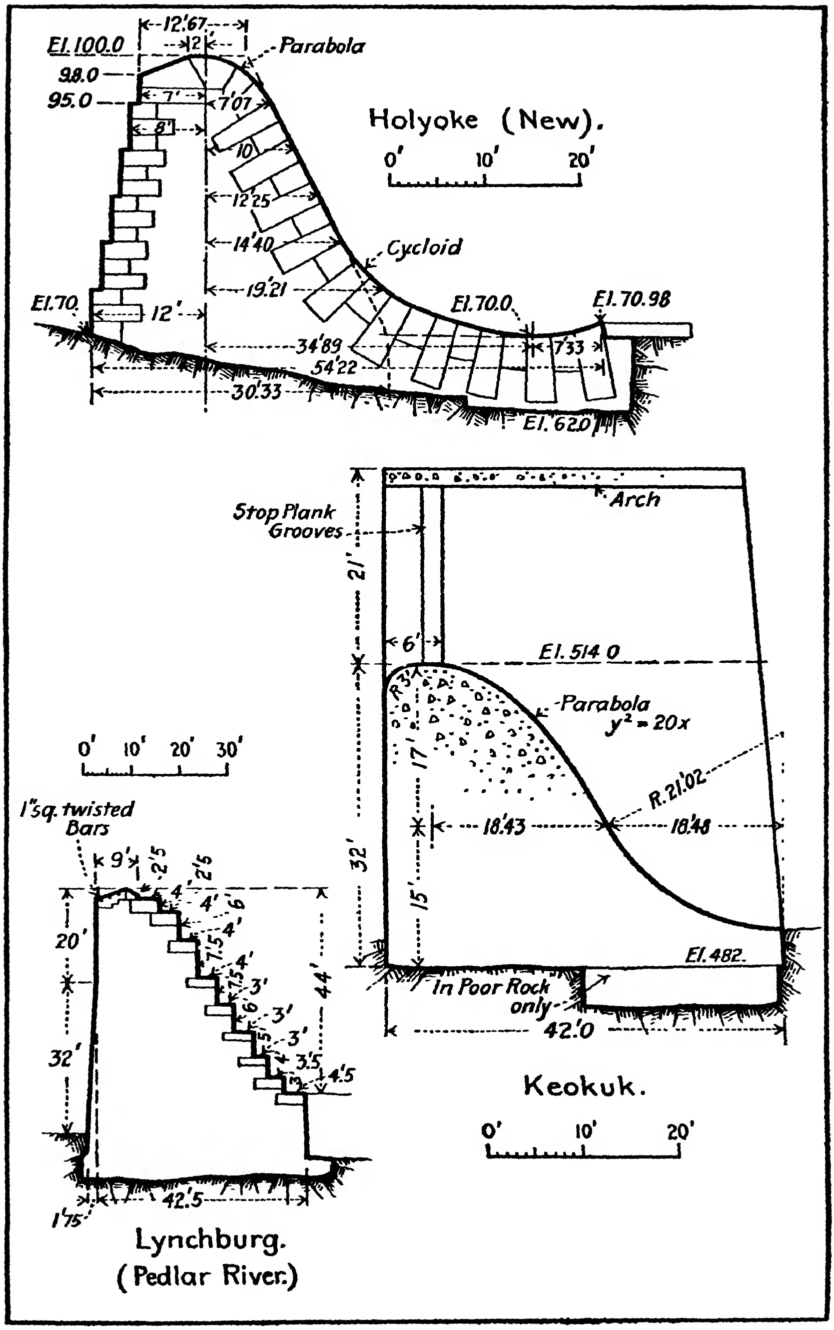


FIG. 78.—Overflow dams.

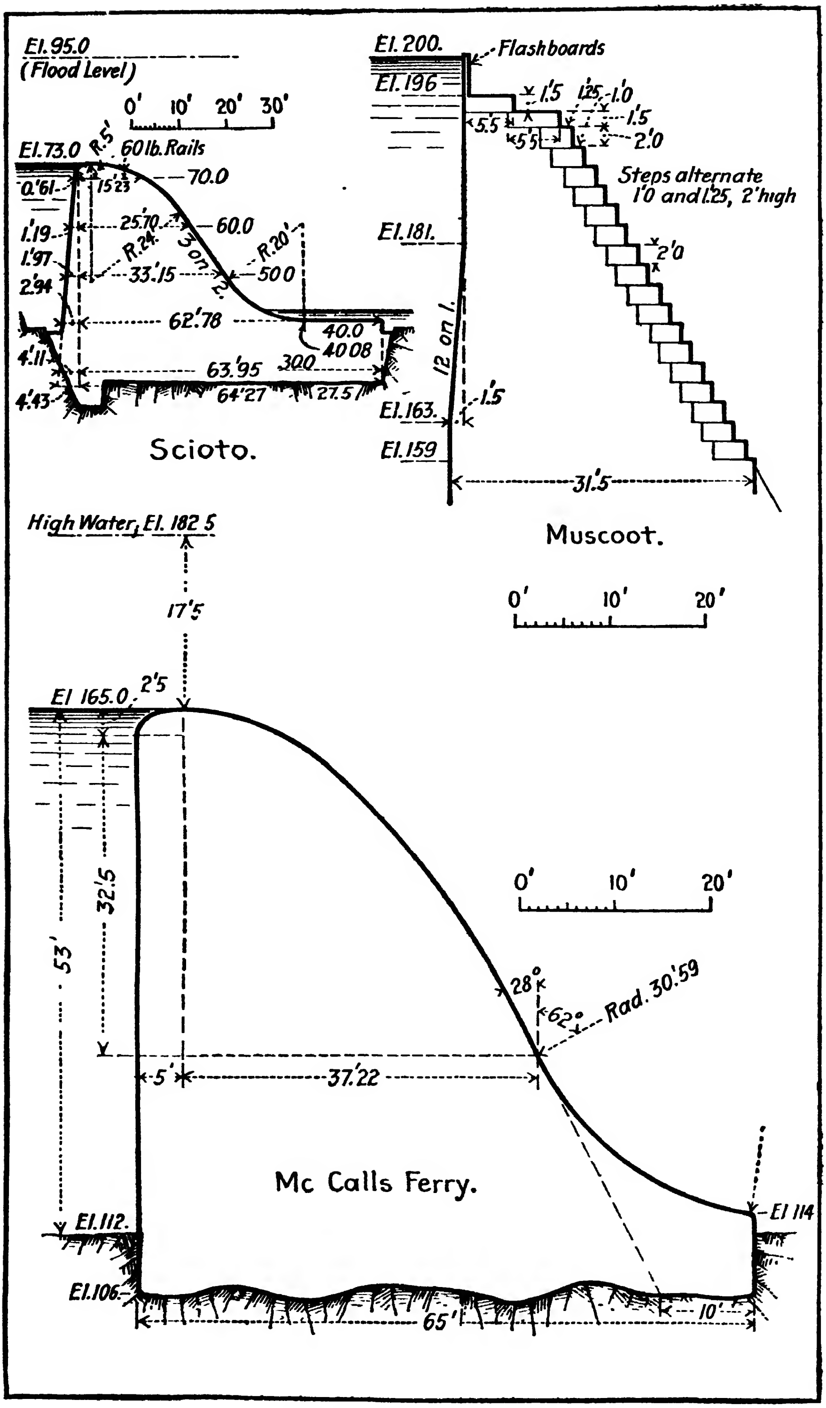


Fig. 79.—Overflow dams.

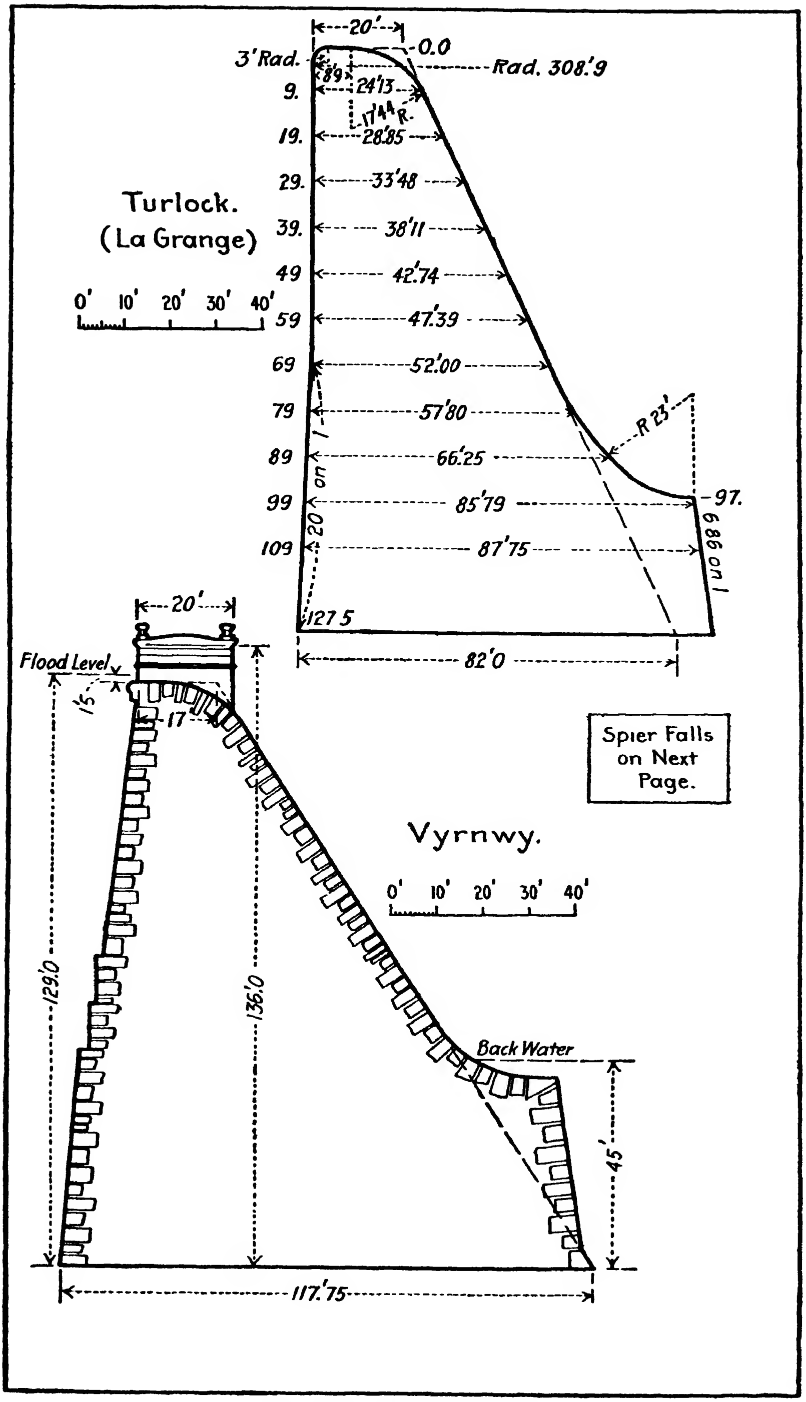


FIG. 80.—Overflow dams.

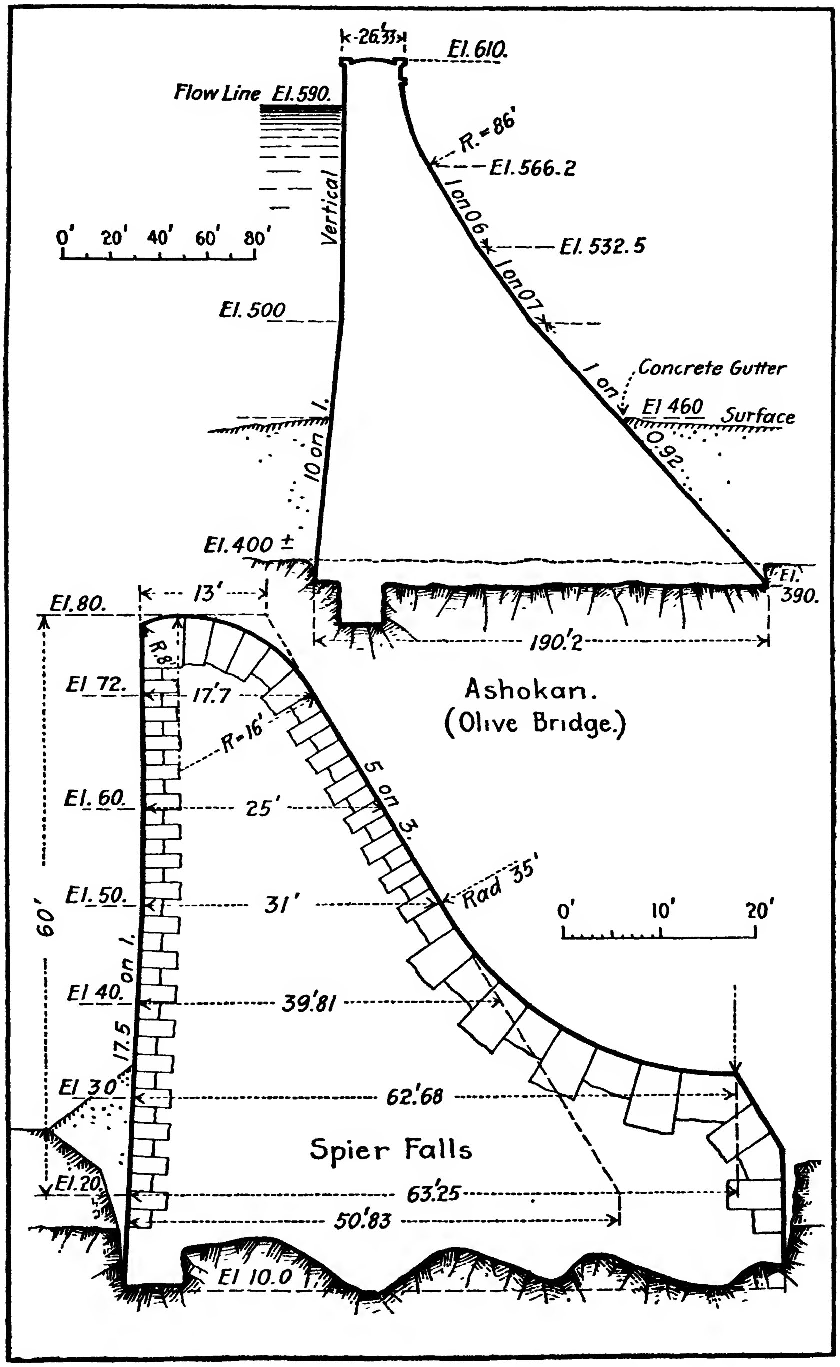


Fig. 81.—Ashokan and Spier Falls* dams.

* For non-overflow portion, see page 187.

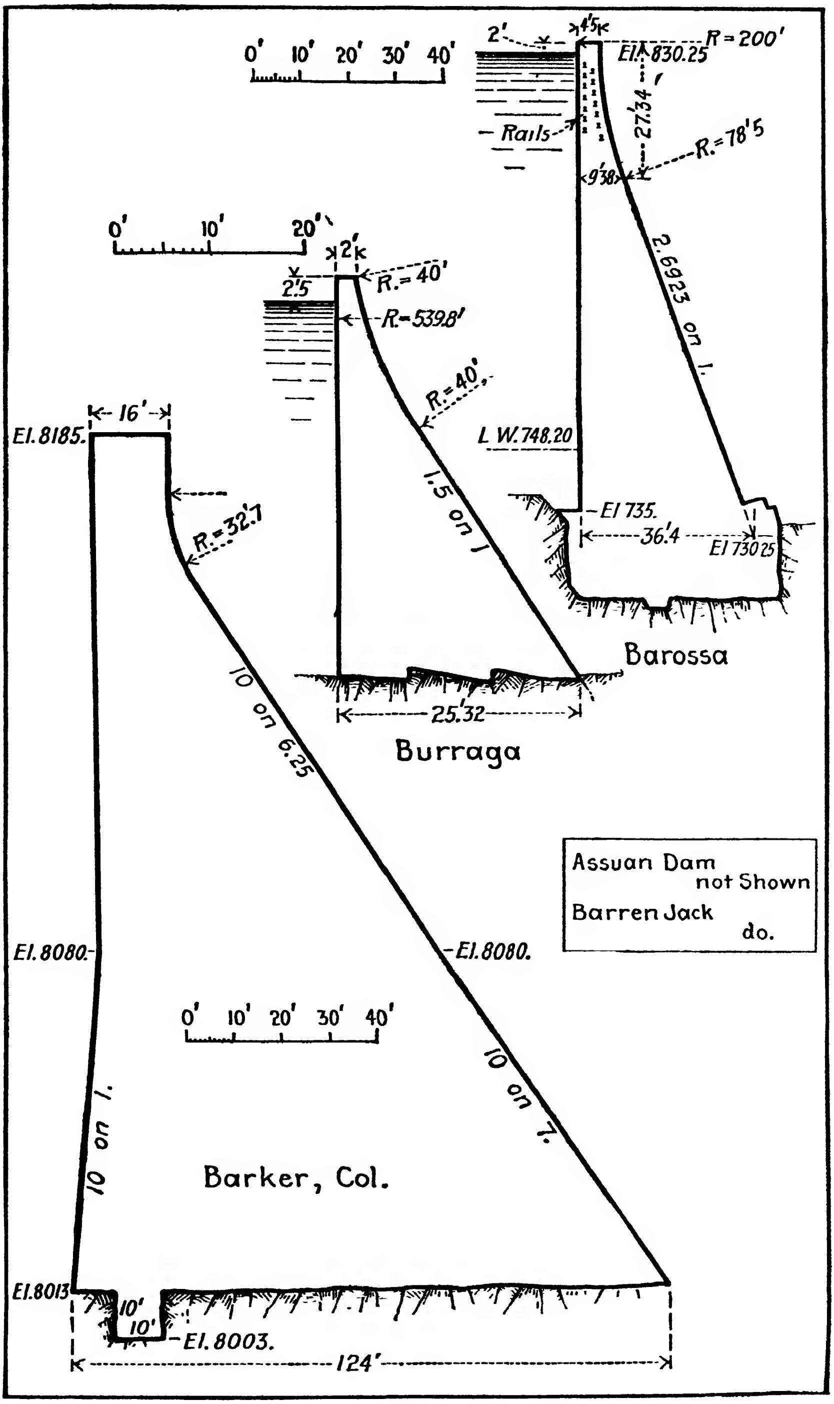


FIG. 82.—Non-overflow dams.

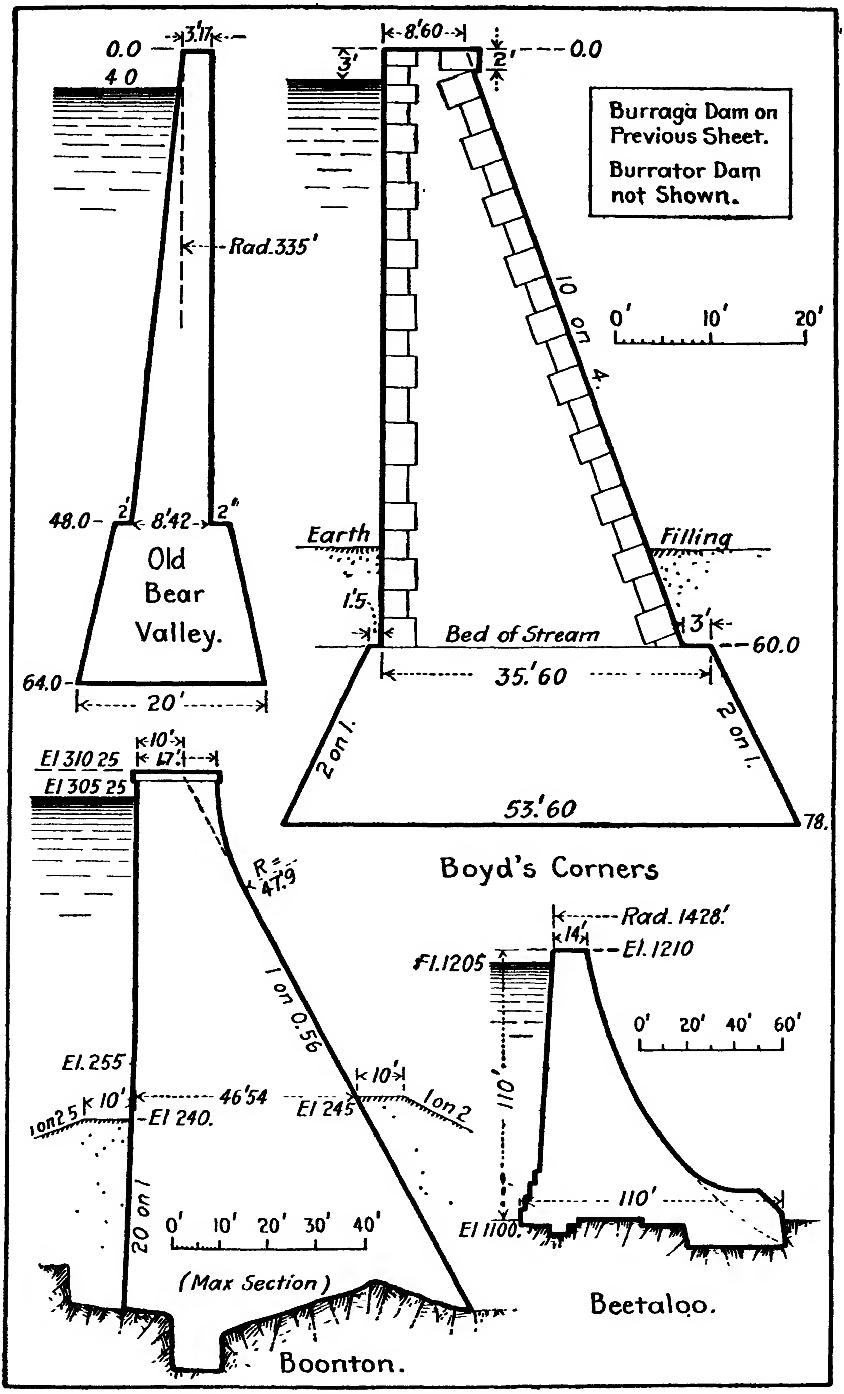


FIG. 83.—Non-overflow dams.

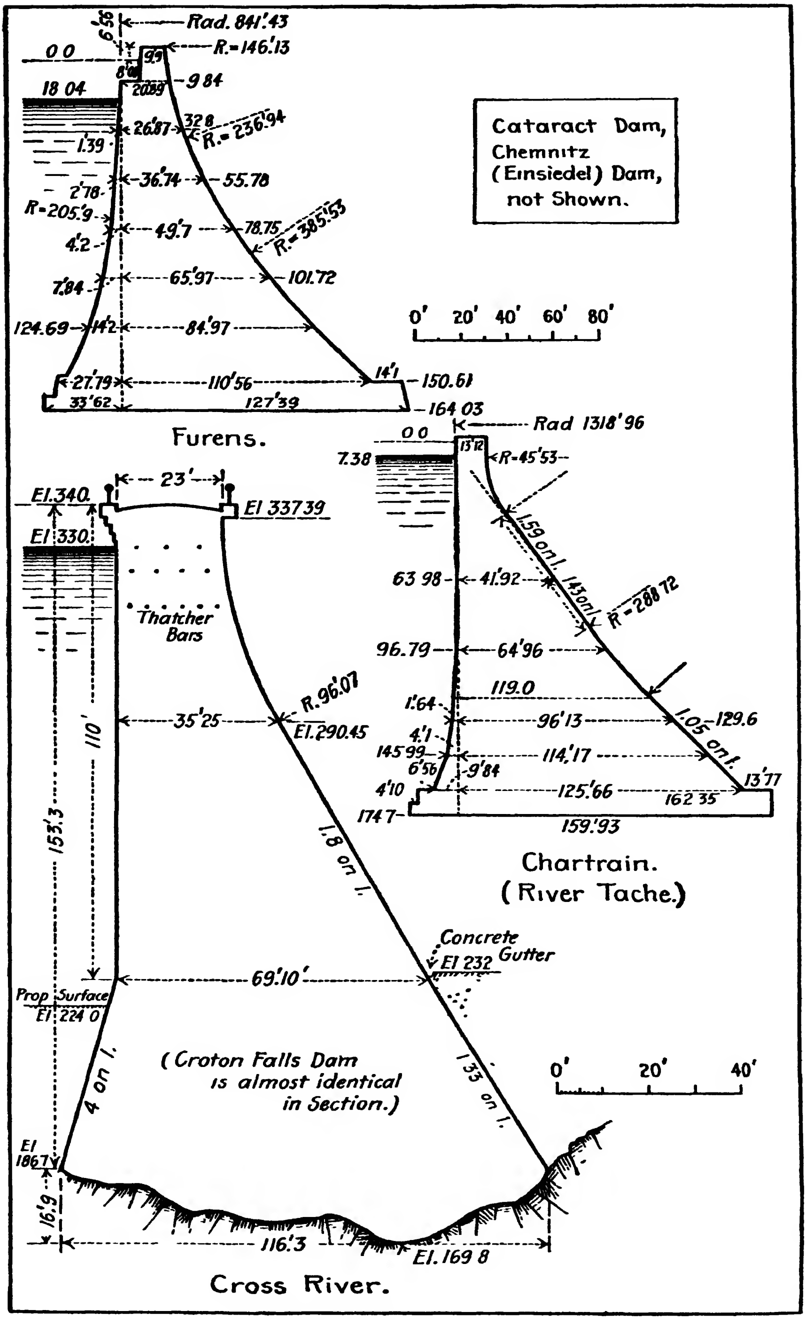


FIG. 84.—Non-overflow dams

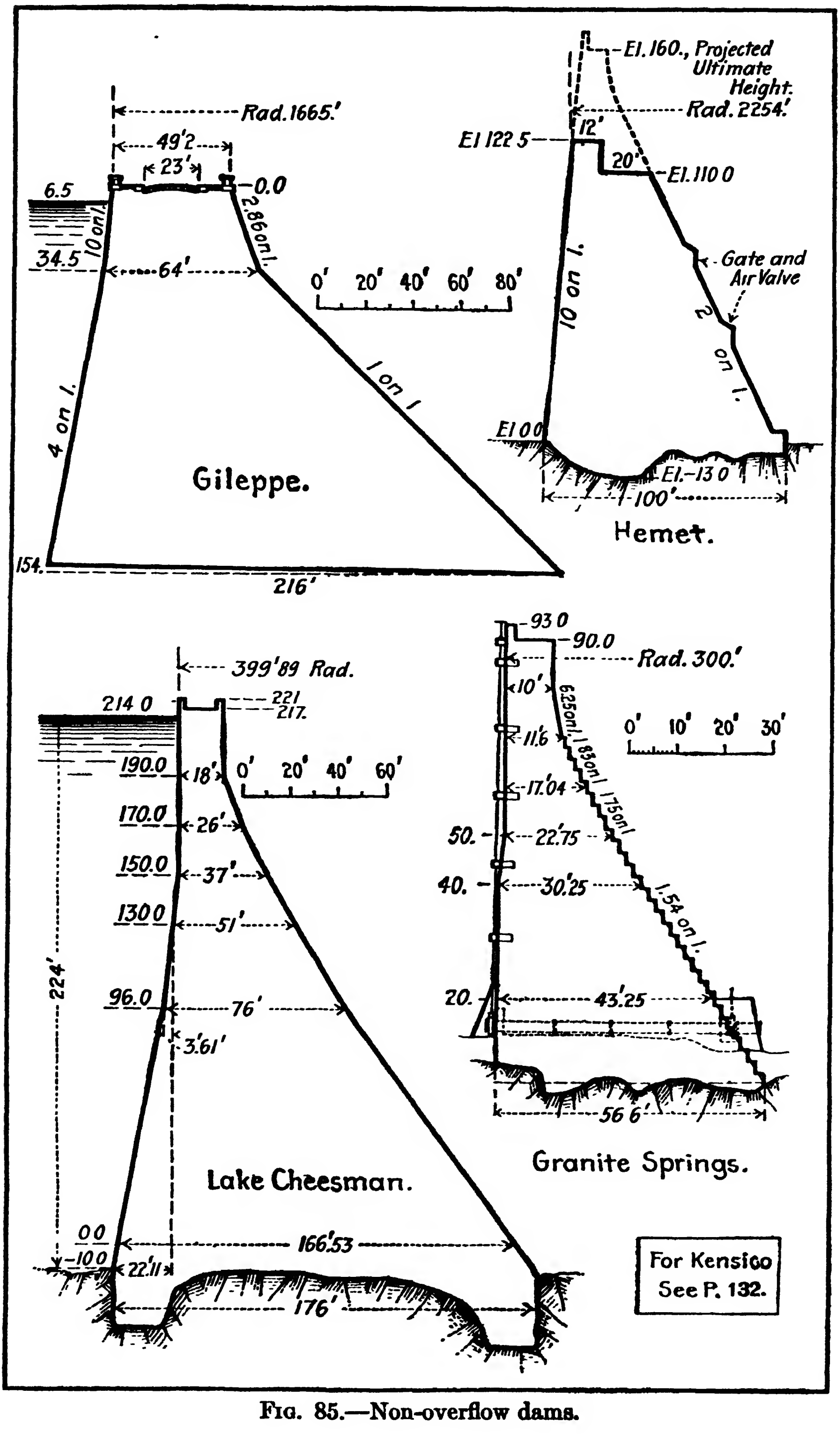


FIG. 85.—Non-overflow dams.

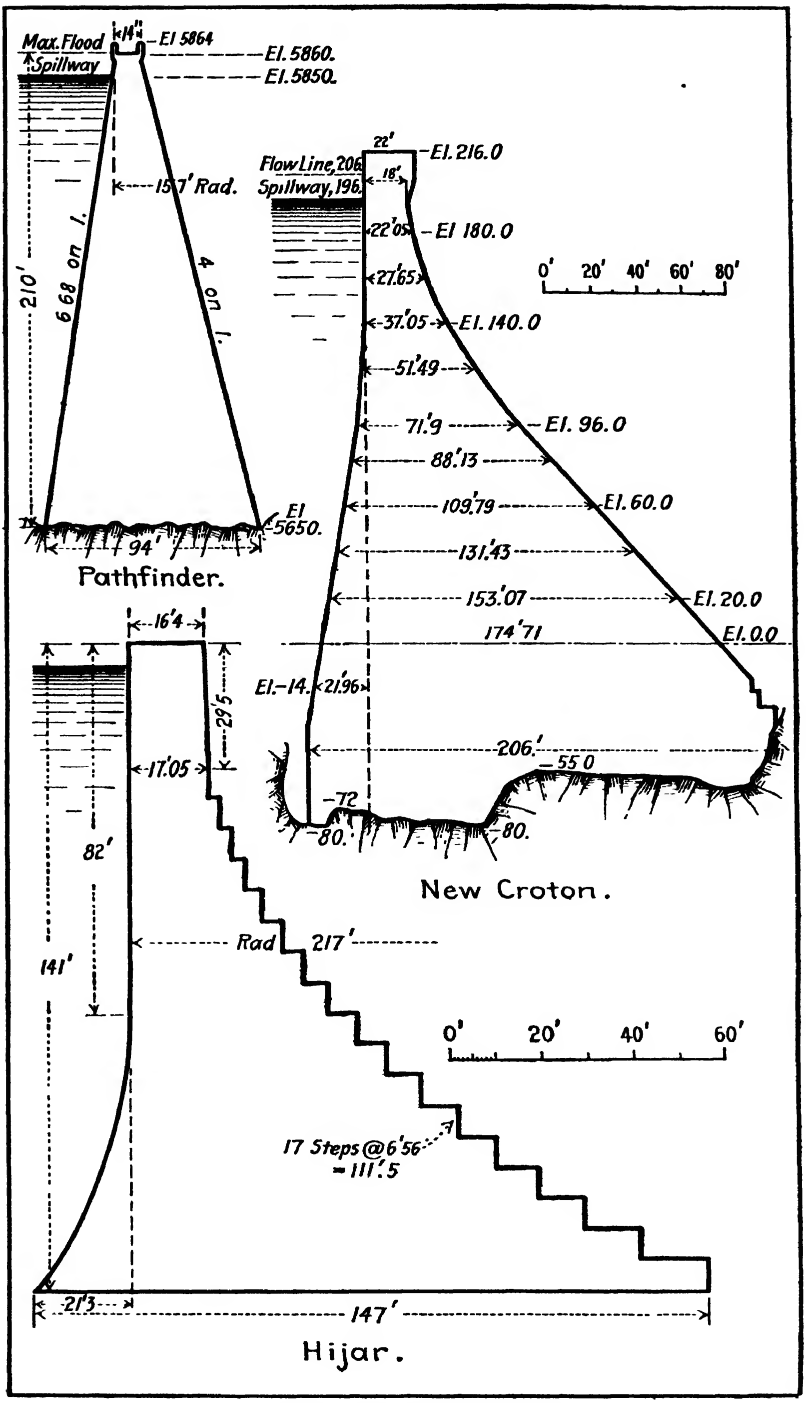


FIG. 86.—Non-overflow dams.

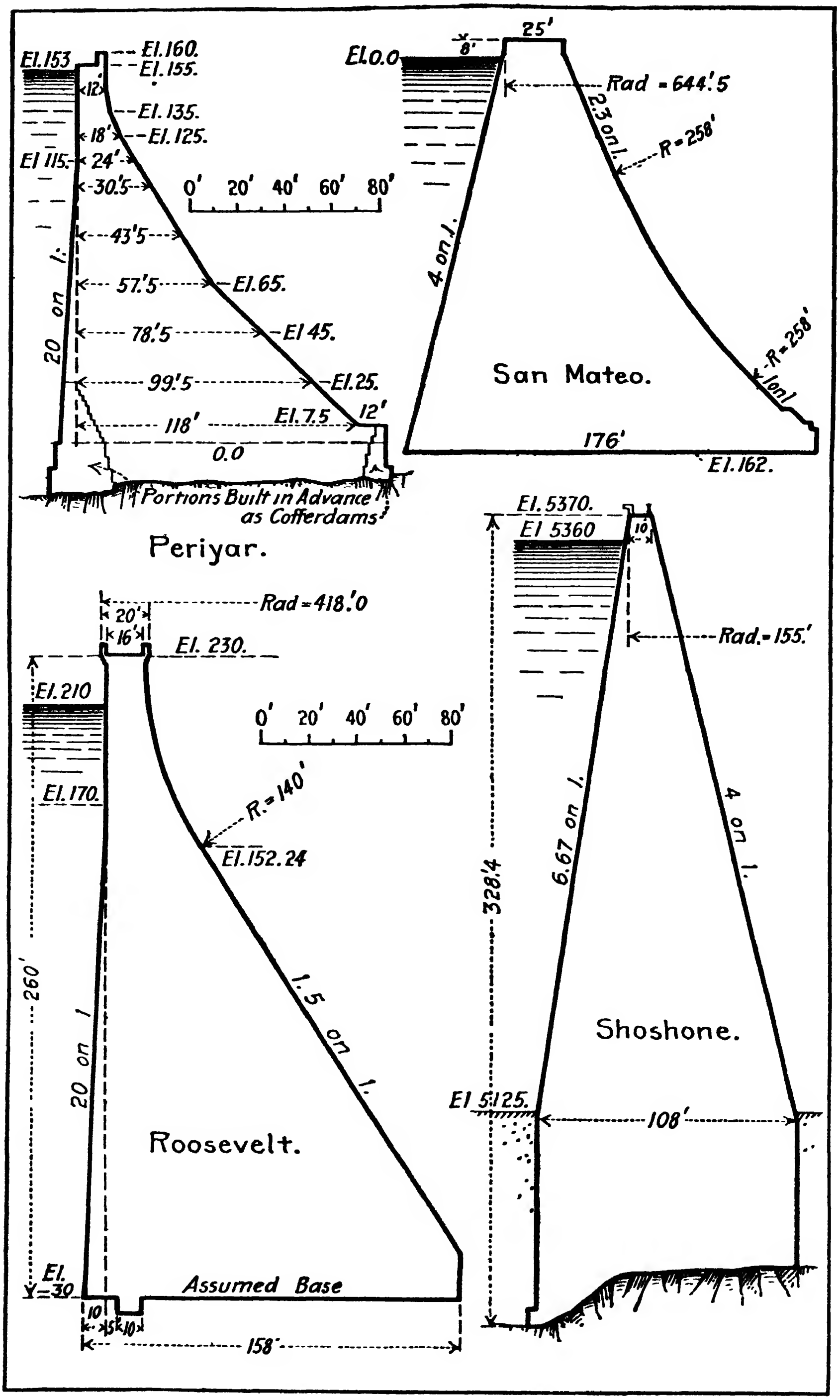


FIG. 87.—Non-overflow dams.

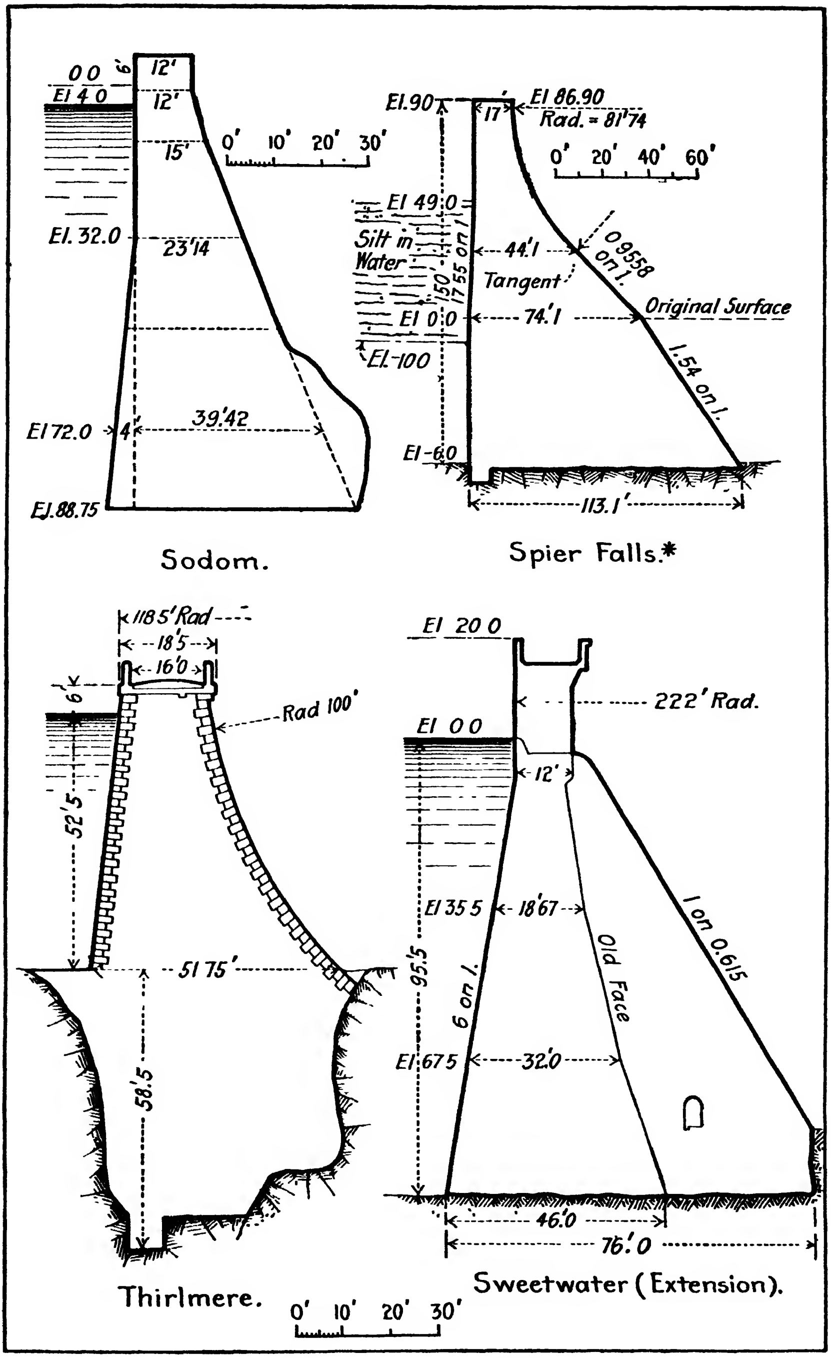


FIG. 88.—Non-overflow dams.

* For overflow portion, see p. 180.

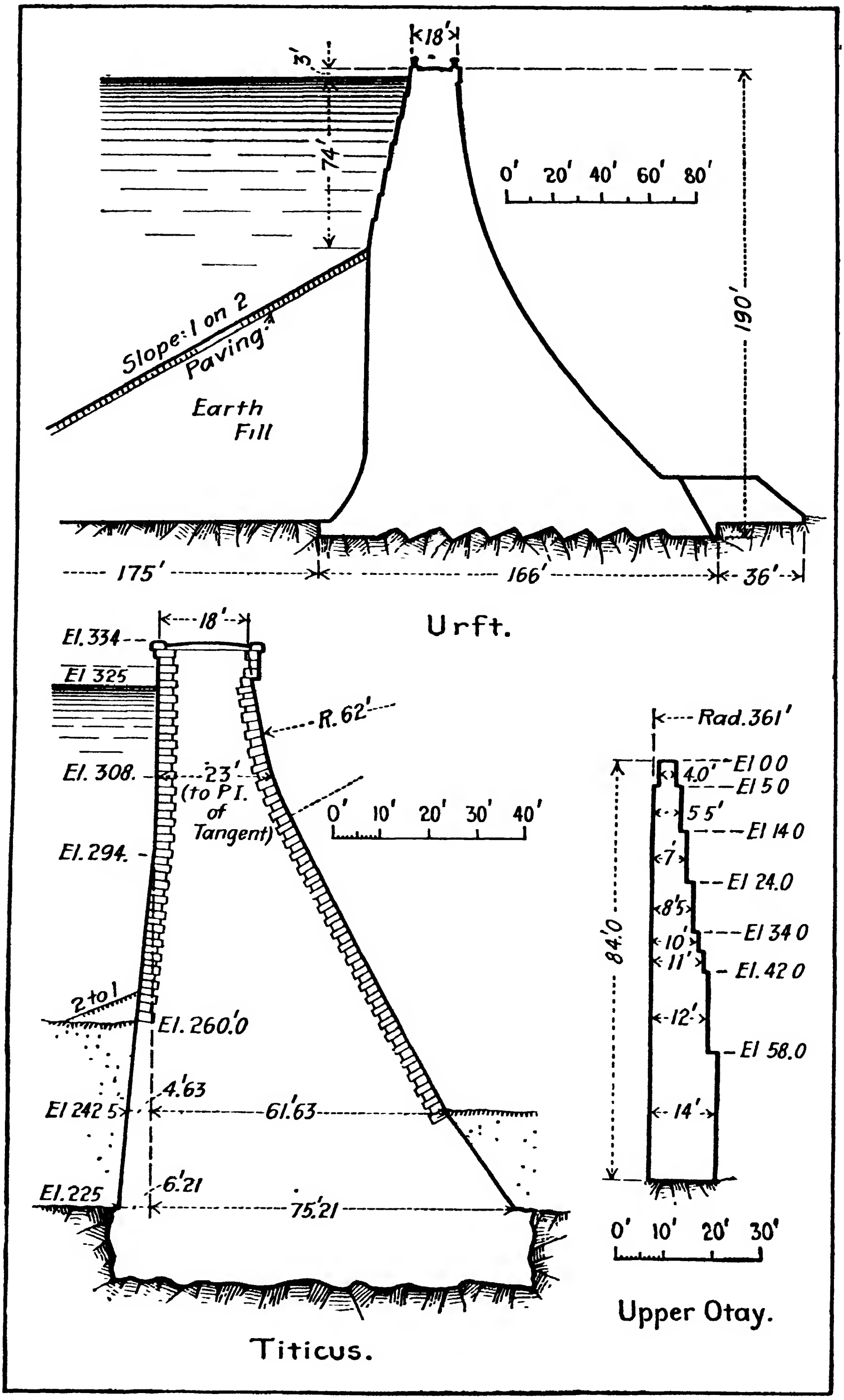


FIG. 89.—Non-overflow dams.

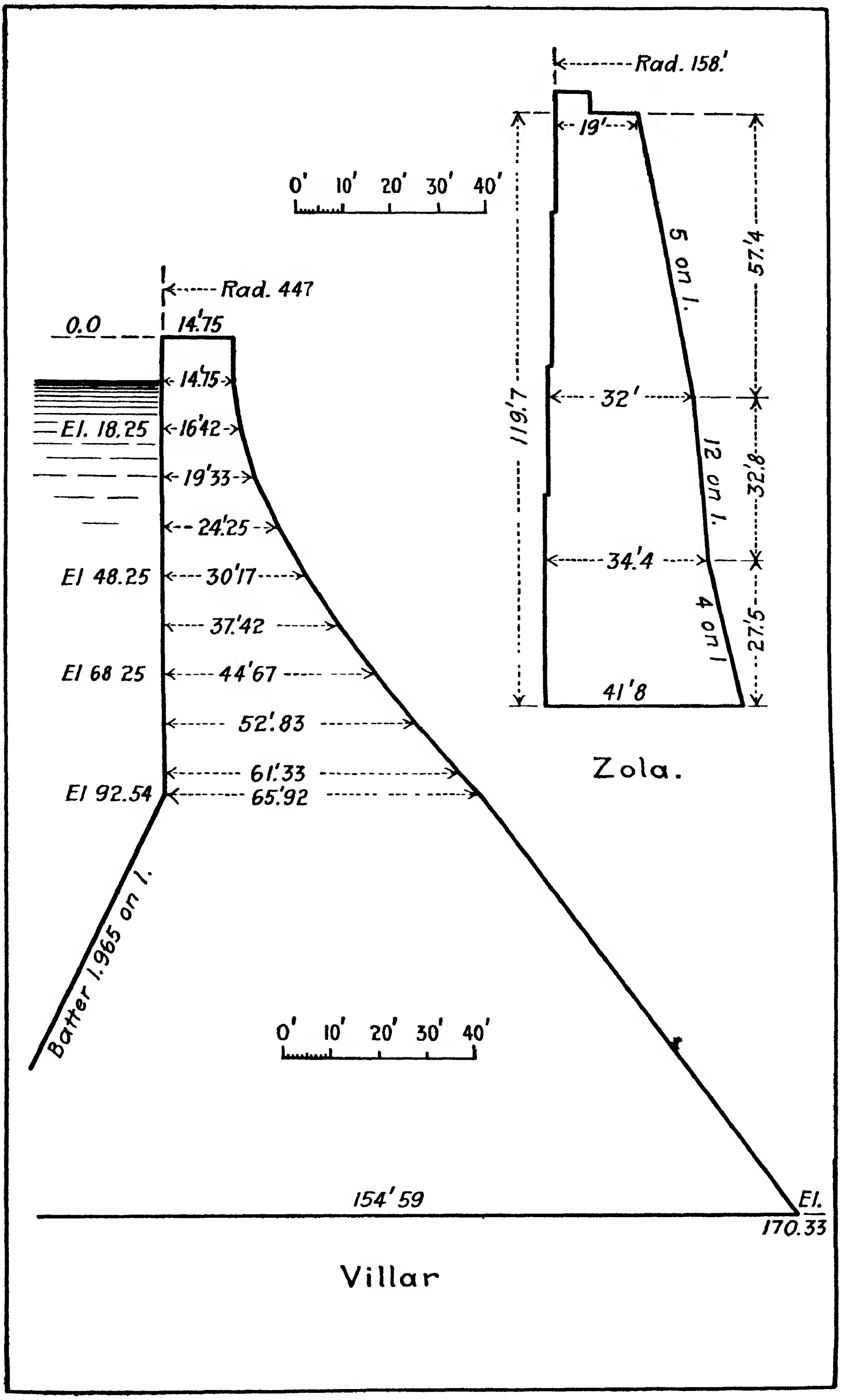


FIG. 90.—Non-overflow dams

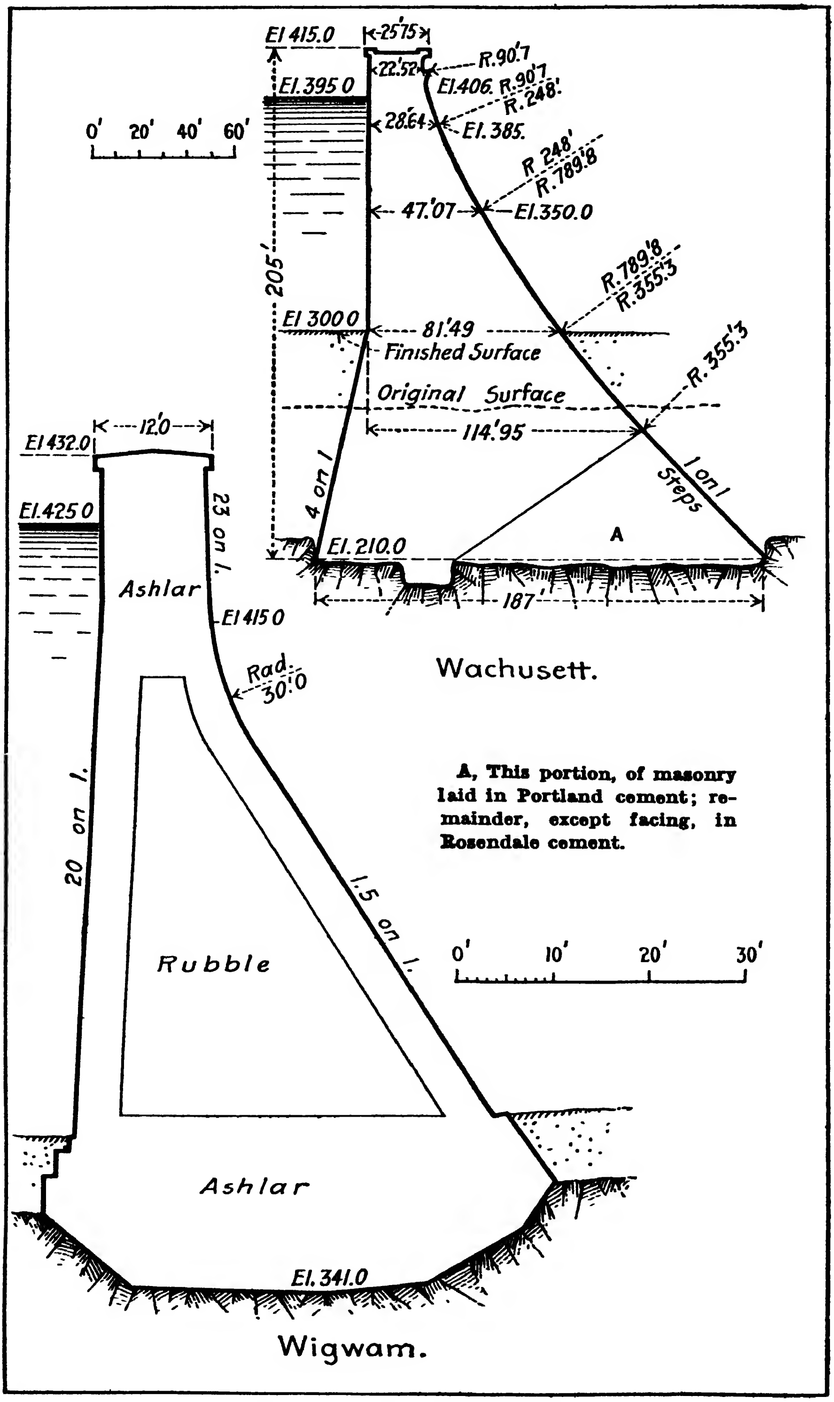


FIG. 91.—Non-overflow dams.

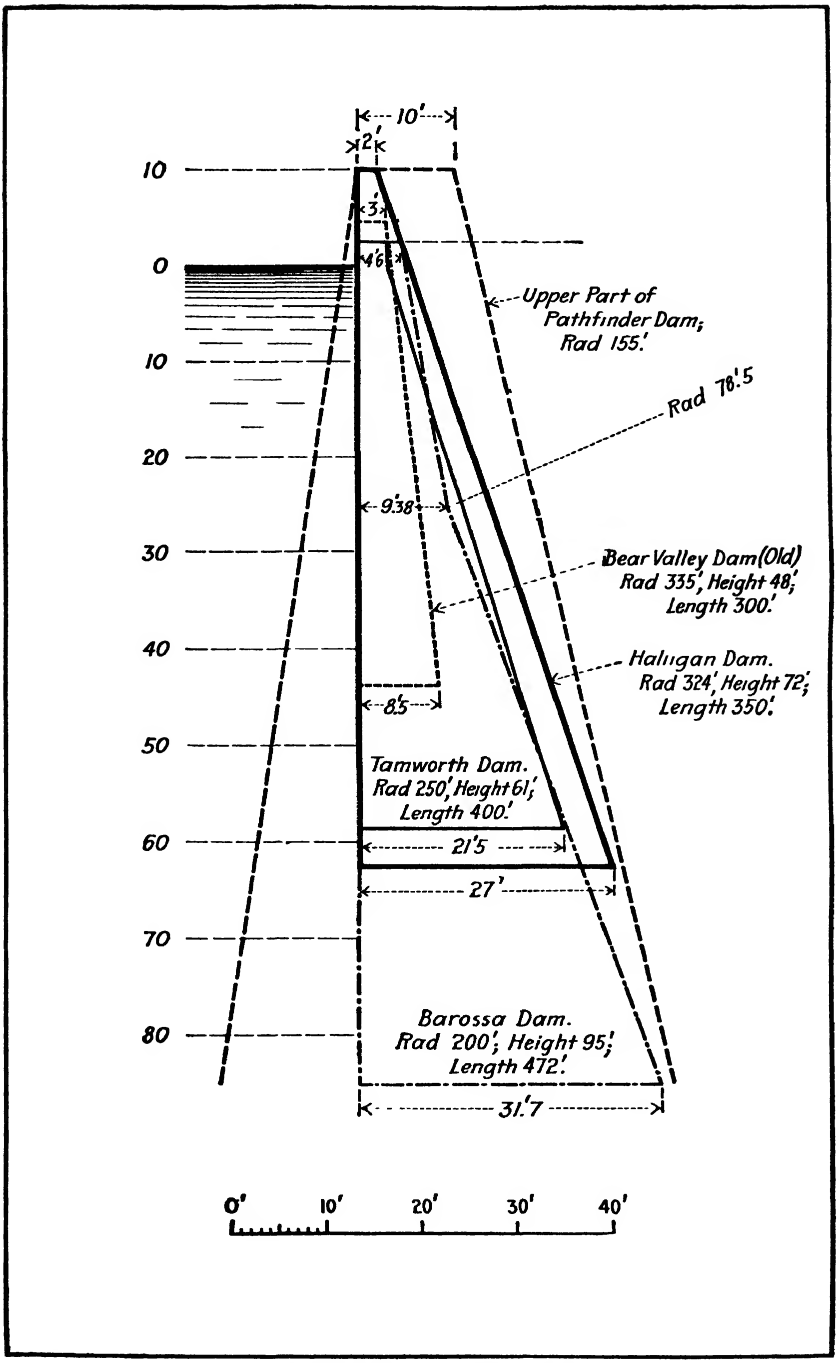


FIG. 92.—Comparison of sections of arched dams.

Radii of horizontal curves are to upstream faces. (T A S C E, Vol 75, 1912, p. 127.)

CHAPTER VIII

ROCK-FILL DAMS

Advantages. A rock-fill dam is an embankment of loose rock fragments supporting a water-tight face or containing a water-tight core of masonry, wood or metal, or some combination of these materials, or backed by an earth embankment. Chief advantages are low cost and quick construction. Where the foundation is pervious rock, making an earth dam impossible and requiring a masonry dam of large mass to resist upward water pressure, the free-draining character of a rock-fill makes it especially suitable. Almost complete elimination of cement is a great advantage in inaccessible regions.

Dimensions and Cores. The foundation should be rock stripped of all earth. A cut-off trench is excavated, in which the facing or core is started, wood or metal being footed in concrete or other masonry. Top width should be not less than 10 ft. and hight above extreme high water not less than 5 ft. There should be a liberal spillway. Upstream slope may be 1 on $\frac{1}{2}$, or even steeper if laid up as a dry rubble wall to support water-tight facing. Downstream slope, or both slopes if core is used, may be 1 on 1 to 1 on $1\frac{1}{2}$, and may be rough or laid to a neat face. Some rock-fill dams have steep downstream slopes; others have nearly vertical faces, and some have stepped faces of roughly squared stones. One or both faces may be laid in mortar and faced with concrete. Metal core or facing is usually of steel plates riveted and calked, coated with asphalt, or asphalt and burlap, and supported by concrete walls or embedded in concrete. Plates should not be less than $\frac{1}{4}$ in. thick and ought to be thicker in the lower part of a high dam.

Requisites. Rock-fill dams may be varied widely in shape and details to suit local conditions; the requisites are an unyielding foundation which cannot be eroded by copious seepage; abundant weight to resist the water pressure; a water-tight core or facing of such materials, and so constructed, supported and protected that it cannot be disrupted; stones for the "fill" of good weathering qualities, washed of all earth, hard enough to withstand the weight and pressure and so placed that no material settlement can occur. Descriptions and illustrations of many rock-fill dams are given by Schuyler (Reservoirs for Irrigation, Water Power and Domestic Water Supply), Wegmann (The Design and Construction of Dams) E. R. and E. N.

Details. M. M. O'Shaughnessy* disapproves of sheet steel in the concrete facing of a rock-fill dam, as adhesion of the concrete to the smooth face of the steel would be questionable, and, with reservoir empty and no expansion joints, temperature changes might cause the steel to buckle, thus creating a cavity which might be subjected to hydrostatic pressure. Soil, silt and clay should be excluded from the mass of the dam, but not quarry waste, composed of spalls. If there is a shortage of spalls, break off sharp edges of flat stones, and chink in the cavities with broken stone. The lower slope of a rock-fill dam

* Trans. Am. Soc. C. E., Vol 75, 1912, p. 27.

should be $1\frac{1}{2}$ to 1, or about the natural slope of rock; and a rubble wall, or the hand-placing of rock in this portion of the dam, is an unnecessary refinement and expense, except for esthetic purposes. The object of reinforcing the concrete facing is to prevent cracks, so that each section will be a unit slab, firmly attached to the stone masonry, but free to move at the joints in response to any temperature or settlement stress. Anchor rods through the masonry form an effective bond between it and the reinforcing rods, and are also useful in construction in fixing accurately the position of bars.

Escondido Dam, Cal., facing has two layers of redwood planks laid longitudinally of the dam, breaking joints both ways, calked with oakum and smeared with hot asphalt. In lower third, the planks are 3 in. thick, 2 in. in middle third and $1\frac{1}{2}$ in. above, and these top planks extend 3 ft. above

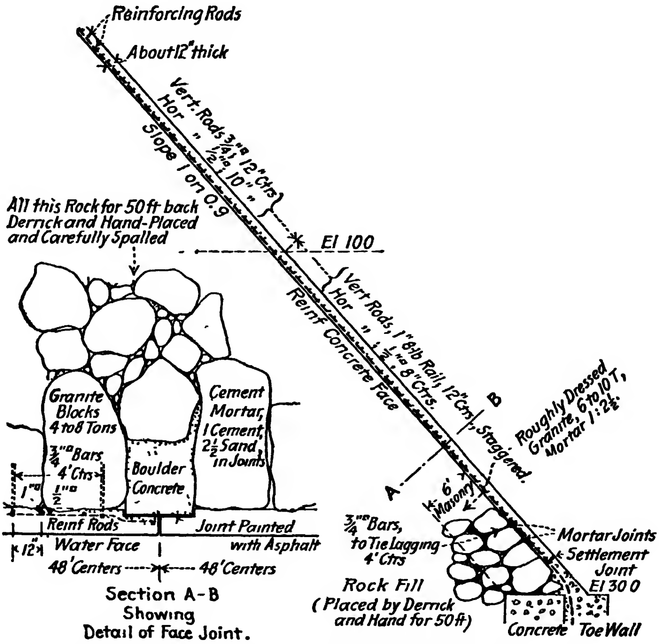


FIG. 93.—Morena dam. Detail of facing on water side.
(T A S C E, Vol. 75, 1912)

top of stone fill. Planks are spiked to 6 × 6-in. redwood timbers embedded in dry rubble wall of the upstream side and projecting 2 in. out of the stone. These timbers are laid up the slope, 5 ft. 4 in. on centers. As the planks were laid, the 2-in. space between them and the stone was rammed full of concrete.

Alfred Dam, Maine, 39 ft. high, 995 ft. long, upstream slope 1 on 1, downstream slope 3 on 1, was built of split-stone rubble laid by hand, with mortar in the three lower courses only; the upstream face is 2 ft. thick, of 1 : 2 : 4 concrete, of which the crest also is formed with maximum thickness of 4 ft. Completed 105 days after beginning of clearing, 90 days after beginning of stone work.

For Morena rock-fill dam,* San Diego, Cal., 180,000 tons of exceedingly hard granite were dislodged from an adjacent mountain side by one "mass shot," at a cost of 4.3 cents per ton. The charge included 1900 lb. of 60 per cent. dynamite, 3500 lb. 40 per cent. dynamite and 33,500 lb. of 7 and 9 per cent. Champion powder.

Table 48. Rock-fill vs. Solid Masonry. Comparison of Roosevelt and Morena Dams

	Roosevelt dam (solid masonry)	Morena dam (rock-fill)
Hight.. . .	284 ft.	267 ft.
Thickness of base..	170 ft.	300 ft.
Thickness of top. . . .	16 ft.	16 ft.
Contents	340,000 cu. yd.	306,000 cu. yd.
Crest length	1080 ft.	550 ft.
Time consumed in building	4 yrs.	5 yrs.
Cost	$3,468,000	$1,100,000

Note that Roosevelt dam has twice the crest length, and that it was built in one year less time. Freedom from uplift is a desirable feature in favor of the rock-fill type. One impervious surface next to the water pressure is the object to be attained in dam construction. Rock can be excavated economically by mass shots at higher levels above a dam without endangering the site, providing the strata are located so that the effects of explosions will not open seams in the vicinity.†

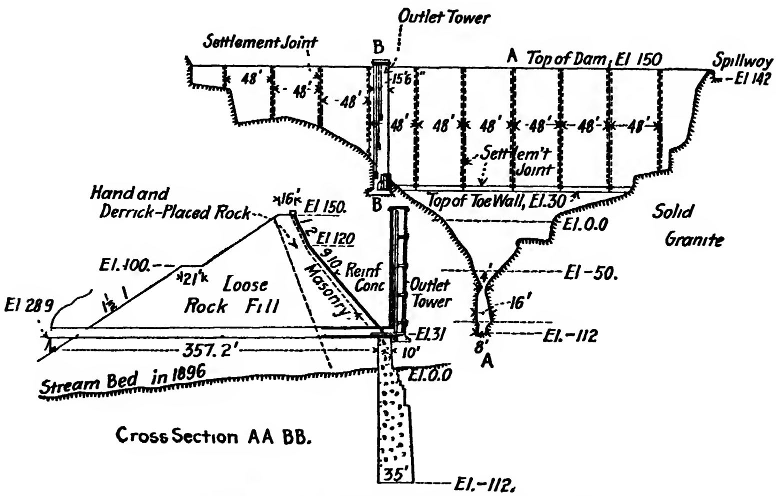


FIG. 94.—Morena dam. Elevation and section.

Lower Otay Dam.* The top of the Lower Otay dam, San Diego, Cal., is 615 ft. long and 15 ft. wide. Rock, from quarries adjoining, was dumped on both sides of the core by Lidgerwood cableways, care being taken not to disturb the core when placing rock adjacent to it. The slopes of dumped rock are

* Trans. Am Soc C. E, Vol 75, 1912, p 27.
† During floods, Jan 1916, Morena reservoir filled within 18 in. of crest without damage to structure. (E R., Feb. 12, 1916)

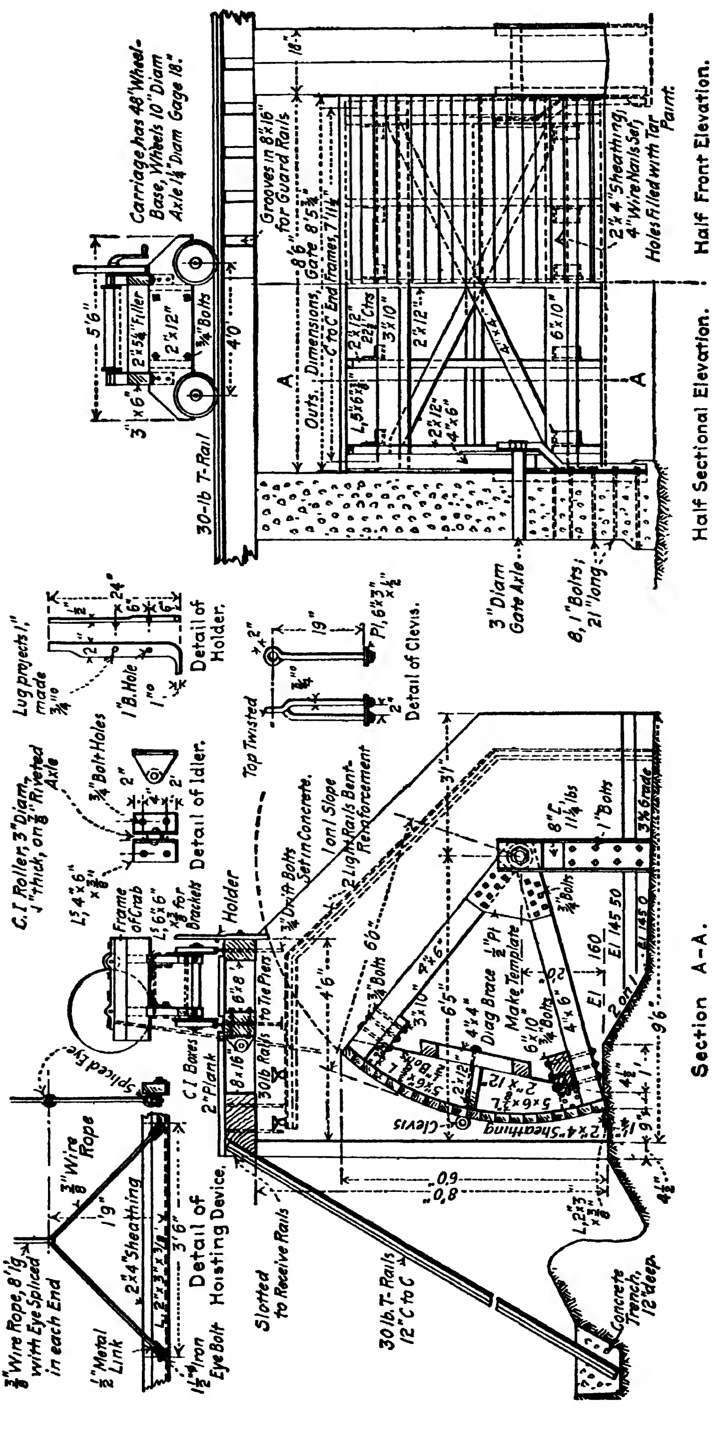


FIG. 95.—Radial spillway gates, Morena reservoir.

a little steeper than $1\frac{1}{2}$ horizontal to 1 vertical. The dam, completed in 1897, had never, up to 1911, been subjected to full head of 124 ft., but in June of 1909, a depth of 109.5 ft., and in May of 1910, a depth of 119.5 ft. caused the saturation, for the first time, of the soft materials mixed with rock from the quarries, and the result was a slight settlement on the upstream side. Additional materials have been hauled to keep the top of the dam level. Through this dam there has always been a slight leakage, which fluctuates with the head in the reservoir, as the formation is a seamy porphyry, but it has not yet attained any serious proportions. This is one of the strongest objections to structures of this type, as it is not possible to determine the exact location of any

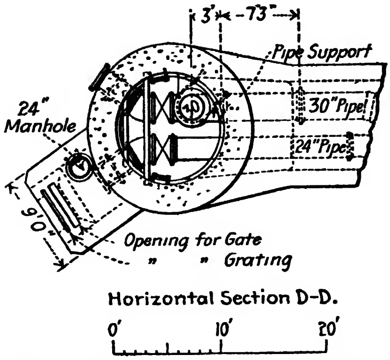


FIG. 96.—Morena dam, gate house.*

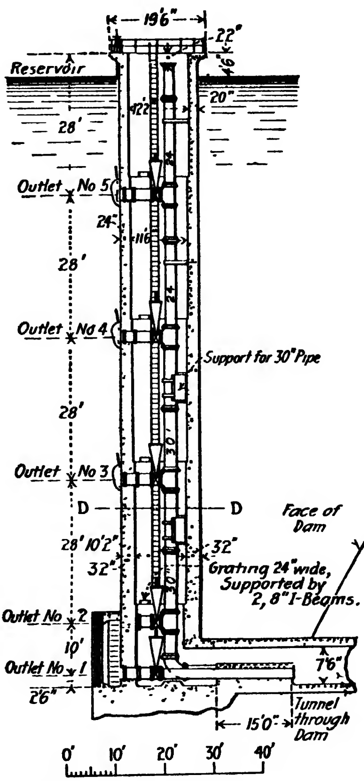


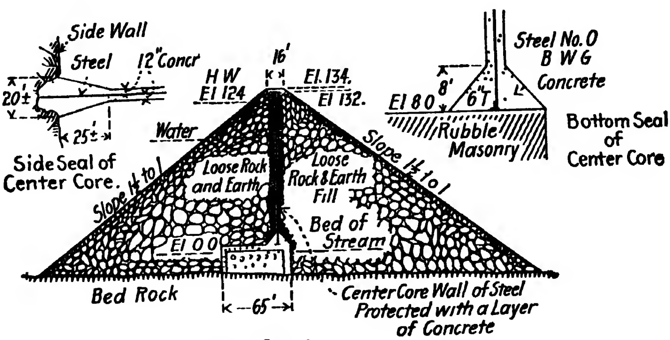
FIG. 97.—Morena dam gate house.

leakage without endangering the dam by prospecting in the mass near the thin core. To obviate serious cracking of the skin which might be caused by settlement, grooves, 3 ft. square, and 48 ft. from center to center, were placed in the masonry, and into these concrete was poured, and brought to an absolutely smooth surface, the general average plane of which projects 3 in. beyond the general face of the masonry. In the future it is proposed to place reinforced concrete slabs, averaging 1 ft. in thickness, on the face of the masonry and attached to it by $\frac{3}{4}$-in. iron rods, 4 ft. center to center. These

* This is a concrete tower in reservoir at heel of dam, see Fig. 94.

slabs will be joined with $\frac{3}{4}$-in. water-tight settlement joints, poured with sand asphalt along the center line of, and resting on, the concrete in the grooves.

Failure of Lower Otay dam* occurred Jan. 27, 1916, after 15 days' run-off from the watershed had raised water level 36 ft.; at failure depth of water was 130.8, as against previous maximum of 119.5. Depth of overflow over top of dam at failure was between 4 and 6 in. The first failure was noticed at the top near the center, and at about the same time water was seen to pour out of the downstream face in considerable volume. One eye-witness asserts that one large boulder was washed out at about mid-hight near center of



Cross Section.

FIG. 98.—Lower Otay dam.

structure. There is no indication that the structure slipped along its foundation or that there were excessive leaks or blow-outs before overtopping.

An examination of steel core exposed after failure showed it in good condition after 20 years' service. Investigation of records shows that a protective coating (coat of hot asphalt, burlap covering and a second hot asphalt coating) had been carefully applied during construction. Sections of this steel plate were found 10 miles below dam site.

Other streams in the vicinity gave record run-offs.* San Diego river discharged 68.6 sec.-ft. per sq. mi. from drainage area of 434 sq. mi. A gaging taken on the San Luis Rey before the peak showed a run-off from 578 sq. mi. of 95 sec. ft. per sq. mi. On Jan. 27, run-off above Sweetwater dam was estimated to have been 33,000 sec. ft.—twice any previous flood. (Based on drainage area of 186 sq. mi., run-off per sq. mi. = 177 sec.-ft. Compare with p. 21.) The peak lasted 4 hr.; before it arrived, the blow-off valves in the Sweetwater dam were opened and carried off 750 sec. ft. Despite this precaution, parapet of dam, see p. 187, was overtopped and withstood an overflow for 7 hr., max. 3.5 ft. above parapet. During this time, earth fill at north end of dam was overtopped and a 75-ft. opening made.†

* E. R., Feb. 12 and 19, 1916.
† Conditions on these streams are favorable to floods: Soucres are in hilly regions of considerable annual snowfall that runs off rapidly under a warm rain. Yearly rainfall on the watersheds ranges from 9 in. on coast to 60 in. in mountains.

CHAPTER IX

EARTH DAMS OR DIKES

Limitations in Design.* No mathematical formulas nor even general rules can be stated for designing earth dams. Long experience has established some dimensions approximately and some methods of construction. Intelligent selection and use of materials in design, and scrupulous painstaking faithfulness in construction are essential. Usually earth dams are employed where good rock foundations and abutments do not exist. Water cannot be allowed to pass over the top. A wide variety of earths may be used, either alone or in combinations, but nature of materials adopted must largely control design and methods of construction.

Classification. Four classes of earth dams may be distinguished for convenience: (1) Simple embankments, substantially homogeneous. (2) Embankments with core. (3) Embankments with facing. (4) Hydraulic fills.

Homogeneous Embankments. Class 1 should be limited to dams of moderate hight, for which excellent material is available. Higher dams may be so built if for secondary reasons they are to be given excessive thickness and hight and the materials are unusually good, or if consequences of failure caused by unforeseeable conditions would not be serious. Best material is a heavy, naturally impervious, tenacious earth, such as hardpan, unmodified glacial drift, or bowlder till, and some mixtures of small gravel, sand and clay. Some homogeneous dams of moderate hight have been made of sand or of fine gravel and sand. Although seepage was high at first, the embankments became tight. Properly constructed, such dams should be successful, especially if the water to be retained is silt-bearing and excessive seepage for the first year or so is not seriously objectionable. Tightening may be expedited by sprinkling loam or other very fine material into the water.

DAMS AND EMBANKMENTS WITH CORES

Kinds of Cores. In Class 2 cores are usually vertical walls or diaphragms at or near center of the embankment, extending from a trench in impervious material beneath the dam to full reservoir level or a few feet above. Cores may be loam, puddle, masonry, or sheet-piling. The purposes of the core are decrease of percolation, prevention of erosion if the dam should be overtopped by an unforeseen flood, prevention of flow through pervious strata beneath the dam, resistance to burrowing animals, interrupting accidental lines of perviousness which might by percolation be developed into water channels and eroded to destructive dimensions. Rich Portland cement concrete cores best accomplish all these purposes. Rubble masonry has been extensively used. Upstream faces of rubble cores (and of concrete when not made sufficiently impervious) have in numerous cases been plastered with rich cement mortar. Where

* See also "Designing an Earth Dam," J. B. Hays, Proc. A. S. C. E., March, 1916.

the dam site is underlaid with pervious sand or gravel of such depth, or so wet that a trench cannot be excavated for a core wall to an impervious stratum, wooden or steel sheet-piling may be driven to the impervious stratum or to sufficient depth to make an effective cut-off. Piling must be very carefully driven so as to be tight; coarse gravel or bowlders may render it impracticable and force the abandonment of the site, or other serious change of scheme. Top of piling, in the trench, should be very thoroughly embedded in the bottom of the masonry, puddle or loam core. In some instances piling may be extended to full hight as the only core; but for several reasons, including uncertainty as to permanence, sheet-pile cores should not be used in important permanent dams, at any rate, not above permanent lowest ground-water level.

Miller Sheet Piles. For North dike of Wachusett reservoir, under high portions founded on sand, a large trench with natural side slopes was excavated to very fine sand or other nearly impervious material, and where borings indicated that underlying material at depth might be somewhat pervious, sheet-piling was driven below the bottom of the trench. Where length of piles exceeded 30 ft. they were built up of three thicknesses of 2-in. spruce planks spiked together to form a tongue on one edge and a groove on the other. For driving these, water under pressure was discharged through a special iron nozzle at the point of each pile, to which a $3\frac{1}{2}$-in. or $2\frac{1}{2}$-in. hose was so connected that it could be readily detached and withdrawn after the pile had been driven. Hammer of pile driver was used merely as a weight to overcome friction against guides and pile previously driven. Lengths 45 to 67 ft. Greatest number driven in a 10-hr. day was twenty-four, averaging $16\frac{1}{2}$ in. wide and 46 ft. long. Piles less than 30 ft. long were 4-in. spruce sheeting with hard pine splines, driven by aid of water jet through 1-in. hose. Methods and piles were devised by Hiram A. Miller, department engineer in charge, and were patented.

Loam cores should be started in relatively wide trenches excavated to rock or into impervious earth. If rock is seamy, the seams should be cleaned out and thoroughly filled with concrete or grout; pervious rock should be grouted or plastered with rich cement mortar. To prevent water passing along the rock surface beneath core, low concrete, rubble or brick cut-off walls should be built wherever necessary, to extend up into the loam. Loam should be very fine grained, free from stones, roots and vegetable débris; it should be spread in thin horizontal layers (3 to 6 in.) and very thoroughly compacted. It should be forced tightly against the sides of the trench and carried above, with liberal thickness, in the embankment. If the rock on which the core is to be founded is seamy and has only shallow cover of earth, it is sometimes necessary to uncover a wide strip on the reservoir side of core. After scrupulous cleaning, this rock should have all seams grouted or otherwise thoroughly stopped, and its whole surface, if pervious, covered with rich cement mortar or grout.

Puddle cores should be of carefully selected materials, faithfully mixed and compacted; should start in trenches excavated to rock or into impervious earth; should be carried 2 to 4 ft. above full reservoir level, and have a top width of about 5 ft. with side slopes of 10 on 1 to 1 on 1. Rock surface should be cleaned and seams made tight. With slopes approximating 1 on 1 the puddle core really is the dam; the outer portions of the embankment serve principally to protect the puddle from waves, frost, etc. Thickness necessary

for puddle core depends largely upon character of the other materials in the embankment and conditions and methods of construction; at the base of the dam it should rarely be less than one-third the depth below full-reservoir level; below the ground surface, in the trench, the thickness may be materially reduced. Some engineers prefer to step the sides of the puddle core, or to step the reservoir side and make the other substantially vertical. Whatever kind of core, if any, be used, precautions must be taken to prevent water under any possible head flowing down along the side of the core and beneath it. A layer of puddle has sometimes been extended from the core, beneath the embankment on the reservoir side and over a portion of the reservoir bottom, to aid in preventing percolation beneath the dam. For further information on puddle see also p. 613. British engineers favor clay puddle cores rather than masonry. The core is placed along the center line of the dam, vertically over the puddle trench. Rankin states that thickness of base of puddle wall should be about one-third the hight, and thickness at the top about one-half the base. A puddle wall is used in three positions: (a) in the center of the dam; (b) on the upstream slope; (c) between these two. The advantages of (a) are: (1) vertical continuation of the puddle trench; (2) minimum quantity; (3) protection from disintegration by wind, water, burrowing animals, etc. The disadvantages of (a) are: (1) distortion or rupture by settlement; (2) difficulty of repairs; (3) upstream embankment may be saturated to an undesirable extent. The advantages of (b) are: (1) settlement is uniform with upstream slope; (2) ease of repair; (3) constitutes a water-proof covering. The disadvantages of (b) are: (1) more material required than for (a); (2) subsoil water may saturate the dam; (3) exposure to action of waves, sun, and burrowing animals; (4) lack of stability at ordinary slopes of dams.

Clay is preferably used as a core. The pressure of surrounding masses forcing out surplus water, leaves clay in such condition that it will neither absorb water nor allow percolation. Surface clays make better core than deep-seated clays. J. D. Schuyler claims that fine sand, with finely pulverized non-plastic material, is preferable to clay for an hydraulic-fill dam, as it does not shrink and is impervious to water. A combination of clay and fine sand is still better. Prof. Carl Hilgard found that the addition of 0.1 per cent. carbonate of soda will make almost any earth available for puddling. Clemens Herschel claims that pure "binding" gravel makes a more waterproof stop than clay. When puddle paving is used, if sprinkled with saltpeter, KNO_3, before rolling, it will not crack nor wash off. The test for puddle dams used in India is to dig a hole 2 ft. square through 1 week's work, fill with water, and note the time it takes to disappear. Good puddle, 4 days after depositing, should cut like cheese and show no air spaces.

Puddle, free from lumps or stones larger than 2 in., if laid in 6-in. layers, will be practically water-tight, but cannot be relied upon to prevent inroads of animals. A layer of broken stone, 6 in. thick, on the lower slope, has been found a sufficient protection against burrowing animals. Puddle of 80 per cent. gravel and 20 per cent. clay is less easily attacked than pure clay. Fine sand in interior of an earth dam has the advantage that animals cannot burrow in it, due to its lack of cohesion. Clemens Herschel says: To be safe for an indefinite period, an earth dam should have somewhere in its cross-

section a durable element, or member of construction, capable of resisting attacks of burrowing animals, not infrequently experienced from upstream as well as downstream side. (Eng'g Contr., Jan. 18, 1911.)

Concrete cores should be of rich, dense, strong Portland cement concrete, carefully placed to avoid honeycombed and porous spots. If practicable, in the foundation trench, the concrete should be compacted against the well-trimmed sides. In loose running earth it will be necessary to give the trench wide flat slopes and start the wall forms on the rock; the trench should be of liberal bottom width and most carefully refilled with compacted embankment material. If trench has been sheeted and braced, so much bracing as feasible should be removed as concrete is placed. No wooden or hollow metal braces should be left to pierce the concrete transversely. Concrete should be packed tightly against any sheeting which must be left in place. A longitudinal groove 3 in. to 6 in. deep and 6 in. to 12 in. wide should be formed in the middle of the top of the concrete placed any day, and, in its vertical face, a V-shaped groove with angle of about 90°.* These joint surfaces must be scrupulously clean, free from laitance, and thoroughly wet when fresh concrete is placed against them. If bolts through the wall are used to support the concrete forms, such bolts should, preferably, be removed, or at least portions for 2 in. in from face, and the holes thoroughly filled with cement mortar. A well-built concrete core should not require plastering. Top width of a concrete core wall should be $1\frac{1}{2}$ to 3 ft., according to importance of the dam. Top should be at full-reservoir level or 1 to 3 ft. above. Each face should batter 20 on 1 to 10 on 1, or may be stepped one or both sides. Thickness at the level below which trench is filled solid with masonry should be $\frac{1}{8}$ to $\frac{1}{6}$ head. Wall should start on clean sound rock or in a trench in very compact impervious earth, or on sheet-piling extending into impervious material. If rock has approximately horizontal seams through which water could flow, a trench should be excavated and the wall extended downward as a cut-off. Grouting under pressure should be done in holes drilled in the bottom of the cut-off trench if seams which would cause trouble are suspected at depths too great for excavating trench at reasonable cost. Importance of dam must determine extent of such precautions. See also p. 164.

A thin core wall in a high earth dam is not advisable, as settlement of the embankment is likely not to be uniform and may reach the wall. Concrete core walls may be carried to hights of many feet ahead of the embankments, but if this is done, the embankments must be built simultaneously on both sides with difference of level not greater than a few feet; otherwise the walls might be pushed over. Since neither horse nor steam rollers can compact the earth close against the wall, this portion of the embankment on each side must be very carefully rammed by hand or very thoroughly puddled so as to be homogeneous with adjoining parts of the bank and in tight contact with the wall.†

Cores are in many cases built too light. E. Sherman Gould's rule was to give a core wall a bottom thickness equal to one-quarter the greatest depth of

* *A raised key*, or tongue, is preferred to a groove by some engineers as being easier to make and clean. On a horizontal surface especially, cleaning is easier. These remarks and those above apply to much concrete work other than core walls, in which interruptions are necessary.

† On some dikes of Ashokan reservoir special 19-ton rollers had rear wheels so placed as to compact the earth close to the core wall.

water in the reservoir, and to draw in with offsets of 10″ on each face every 10 ft. in hight.

If the concrete core wall goes to rock of reasonably good quality, it is better not to disturb the rock by blasting. At Weston reservoir, F. P. Stearns excavated into rock a very little where rock was loose, or where there were large seams, but the rock was cleared for a distance of 20 ft. upstream from the core wall, all seams filled with mortar, and subsequently covered with an embankment of fine, clayey earth; this 20 ft. was about equivalent to the expected water-level in reservoir; Stearns advocates this as a good rule. There seems to be no good reason why top of core wall should be more than 2 or 3 ft. above normal water-level in reservoir, for a flood which raises water several feet does not last long enough to allow bank to become saturated. The stress at top of a core wall is insignificant compared to stresses at greater depths. Some recommend keeping top width of a core wall to 2 ft., and using material saved for increasing the bottom dimensions. (E. C., Jan. 18, 1911.)

Depth of Core-wall Foundation. All porous material should be removed from the site of the embankment. If a good foundation for a core wall exists 10 ft. or somewhat less below the stripped surface, there appears no good reason for carrying it deeper. It is character of foundation rather than depth which is important. In some cases, however, because of porous materials, it has been necessary to extend cores to depths in excess of 100 ft.

Permeability of Cores. Researches by Croes, Smith and Sweet determined that in an earth dam, with masonry core, there was a continuous water plane, extending from the water surface of the reservoir to the core wall, and on the downstream side to the toe, having a maximum inclination of about 20 per cent., showing that cores are not water-tight, as the dam was saturated below this plane.* The cores in North and South dikes of Wachusett reservoir consist of 6-in. layers of fine loam soil well sprinkled and rolled. Experiments have shown that while the plane of saturation on the reservoir side was at water level, immediately below the core it dropped to a level slightly below the base of the dam. Measurements proved that the water draining out of the dike was not in excess of the natural drainage from precipitation on the area of the dike itself. See also p. 206 and E. R., Nov. 30, 1901.

Hollow Core Wall. A cellular concrete core, consisting of two parallel walls a few feet apart, extending through the embankment longitudinally, separated by vertical and horizontal concrete diaphragms, has been proposed by the Ambursen Company. "The upstream wall has a very substantial foot keyed into rock or other impervious material. Downstream wall need not be keyed in, or at most but slightly. Drain pipes with open joints and of large capacity are led from the core wall at frequent intervals out through the embankment, discharging into gutters on the downstream side; joints are cemented as they approach the toe of the embankment in order that they may not be clogged by penetrating roots. Water which may find its way through the submaterial or come through joints in the core wall, or from any source, is intercepted by the hollow wall and drained away. A dry earth

* Examinations were confined to dams on Croton watershed, N. Y., built before present-day Portland cement concrete was known.

prism is secured for the downstream section of the dam, thus materially increasing stability. Hollow core wall penetrates the heart of the whole mass, is well lit from above and accessible by a permanent ladderway. Openings through the party-walls at various levels give access to every foot of both surfaces. Any leak may be located and, if possible, remedied. If not serious enough to be troublesome, or if incapable of being remedied, it is powerless for harm. Being arranged to be covered by planks in winter, the space is not filled with snow and ice." Concrete is reinforced. Top of wall rises slightly above the embankment, and the downstream wall may be a little higher than the upstream.

PAVING*

Substitute for Core Wall. Embankments with facing are much less common than those with cores. A few dams have been so built and such construction is used for artificial reservoirs largely in excavation. Facing may be concrete, rubble in mortar, brick or puddle; it should be laid on

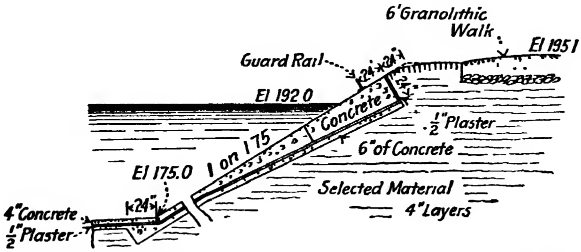


FIG. 99.—Paving, Forbes Hill Reservoir, Quincy, Mass.
(E N., March 13, 1902)

freshly trimmed slopes and ample precautions taken to prevent settlement, cracking and damage by ice, waves or extreme summer heat. Puddle must be protected by paving or riprap. Slope and footing of the facing must be such that sliding and buckling cannot occur. If water in reservoir be rapidly

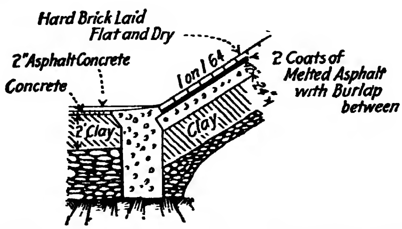


FIG. 100.—Queen Lane Reservoir, Phila. Junction of bottom lining and slope paving.

drawn down, there is danger that back pressure of water in the embankment may lift the facing. Provision for this must be made by drains or weepers, which will not in turn become danger points in other ways.

Pavement and Riprap. Water slopes should be protected with pavement or riprap, extending from bottom to at least 2 ft. vertically above full-reservoir

* See also page 519.

level, and much higher for large reservoirs.* The top above the paving and the downstream slope are usually covered with soil and grassed, as a protection against rain erosion, etc., and for more pleasing appearance. In regions of little rain these parts are sometimes covered with gravel and kept free from vegetation. If field stones or quarry fragments are abundant, the downstream slope is sometimes covered with them giving a pleasing appearance, but gravel or other porous material should be placed under the coarse stone to prevent erosion under the latter by surface drainage. On water slopes pavement is usually of stones, but concrete in slabs or blocks and rubble masonry in mortar are used in some instances, especially in distributing and equalizing reservoirs or sedimentation basins which are to be cleaned occasionally. Stone

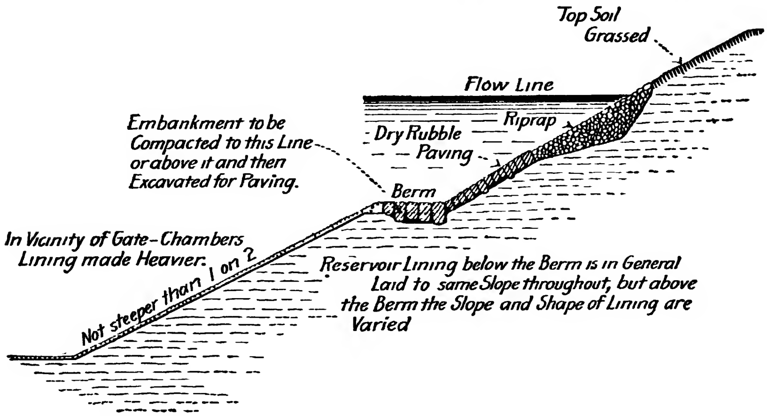


FIG. 101.—Hill View Reservoir, New York City. Typical details of lining. (Variations above berm are for landscape effects.)

paving varies from large rectangular granite blocks, with 1-in. joints well filled with tightly driven spalls laid on gravel or broken stone, to stones of miscellaneous sizes and shapes only roughly laid to slope. Materials at hand, permissible expenditure, nature and importance of dam and similar considerations determine. Likewise thickness of paving may vary from 1 ft. to several. Stones should be hard and of good weather-resisting qualities, especially in regions having severe frost. Heaviest and best stones and greatest care in laying should be used from top of paving to a few feet below full-reservoir level, in order better to resist waves and ice. For this belt, paving of a distinctly better class is frequently used and cheaper paving or riprap below. For best results paving should be laid on a layer of broken stone or gravel from 6 in. to 1 ft. or so thick, to prevent earth being washed out from between the large stones by waves or by water draining out when the reservoir level is rapidly lowered. To resist sliding or displacement by ice it is well to put many long stones in the paving, like headers, extending through the backing. These ought to be well distributed. Berms in long steep slopes reduce tendency to slide. If riprap or a poor class of paving be used, the thickness should be greater in the belt at and near full-reservoir level, since by action of ice and waves some stones will be displaced or moved toward the

* Ten feet, vertically, is not too much for large reservoirs exposed to very high winds.

bottom. Caution must be exercised, however, not to place too heavy a load on the upper portion of a slope if the bank be composed of material which when saturated may slump under the load. On the other hand, slumping of a bank or an excavation slope in fine, water-bearing material may be stopped or prevented by loading with a thick layer of coarse stone; in this case the layers should be thicker toward the bottom. Thick riprap is cheaper and better than paving if large stones can be had plentifully at small cost; it stops waves from gliding up the slope and does not require expensive repairs when displaced. On flat slopes (1 on 4 or flatter), riprap of small stones is thoroughly effective.

If the earthy material for the dam contains many small boulders and cobbles, these may be disposed of cheaply and the stability of the dam improved by placing them in a thick layer on the water slope or on both slopes. If the downstream slope is to be grassed, the larger spaces in the face of the stone slope must be filled with small stones and the whole covered with a layer of clay before placing loam. Such stone layers should be built up with the earth portion of the dam. Paving or riprap is frequently placed after earthwork is wholly or nearly finished.

Wave Action. The inner slope of Riverside reservoir, Colorado, was paved in 1907, with reinforced concrete ($3\frac{1}{2}$-in. mesh, #10 wire) 3 to 4 in. thick. Slope was 1 on 1.5. Wave action from a 4.5-mile stretch* rivals the impact of coastal waves. Paving was laid in 10-ft. strips, with butt joints; no vertical joints. Wave impact thrust water into the butt joints, washed out the sand and undermined paving, causing failure. An inspection showed the reinforcement poorly placed, some of it being entirely below the concrete. The new paving placed where the old paving failed was in blocks, 10 ft. by 15 ft., all joints being stepped. Thickness was 5 in. Where new paving was placed above old, butt joints were used, but breaking joint with the lower course; two layers of tarred felt were placed in the joints. (Eng'g Contr., Apr. 26, 1911.)

Most of the paved slopes of Sudbury reservoir were 1 on 2. Paving is 18 in. thick laid on 12 in. of gravel in which it is thoroughly bedded. Paving was laid by hand, interstices filled with smaller stones, and spalls driven in. Examination 5 yrs. after laying showed that wave action had caused uniform settlement which in no wise injured the paving. Where not exposed to wave action, paving still kept to line and grade.

Erosion of earth embankments by wave action may be greatly retarded by a floating timber boom, anchored about 3 ft. from the bank. Spring Valley Water Co., San Francisco, has for years used log booms anchored a short distance above its earth dams to still waves. Booms last about 20 yrs. (E. R., July 2, 1910, p. 11.)

Hight of Waves. By Stevenson's formula hight of wind waves in feet is $H = 1.5\sqrt{F} + (2.5 - \sqrt[4]{F})$, F being "fetch" in miles.

DESIGN AND CONSTRUCTION†

Width and Hight of Top. Width of top, hight above full-reservoir level, and slopes should be determined together so as to insure ample thickness at and below full-reservoir level. Available materials, size of reservoir and neces-

* See also bottom of page 214

† See also "The Design and Construction of Impounding Reservoirs" by Wm. Watt, Trans. Assn. of Water Engrs., Vol. 13, 1908, p. 119.

sary factor of safety will principally determine. If back of the dam there is a long stretch of reservoir in the direction of persistent winds, friction of wind on the water surface will raise the water-level against the dam as well as making high waves. These may be combined with a flood. The top of a dam should be so high that even tops of waves will not send water over.

Hight above full reservoir should not be less than 5 ft. in small reservoirs and 10 ft. for larger ones. For important dams it is frequently made 15 to 20 ft. Hight and weight above water-level are needed to resist water and ice pressures and maintain compactness to resist percolation. The depth to which frost tends to make the earth in the top of a dam friable and pervious must not be overlooked. Width of top should not be less than 10 ft., except in unimportant dams; requirements for highway may increase this to 30, 40 or even 50 ft. or more.

Slopes and Berms. Side slopes for dams not exceeding 30 to 40 ft. in hight may be uniform from bottom to top—1 on 1½ to 1 on 3 for the water side, and 1 on 1½ to 1 on 2 for the downstream side. Slopes on the water sides of high dams should not be steeper, as an average, than 1 on 2½. Slopes are determined largely by the character of material and the position it would assume when saturated, also by factor of safety desired and importance of dam. For high dams horizontal berms should be used to break the slope at vertical intervals not exceeding 30 ft. and the slopes should be flattened toward the bottom, imitating the slopes of a natural hill. Much flatter slopes have been used by some engineers. Side slopes as flat as 1 on 5 are sometimes used for compacted embankments, and very much flatter in special structures.

Allowance for Settlement. To offset settlement, earth embankments are commonly built to a slight excess width and hight. Slopes are generally trimmed subsequently, particularly if high-class paving is to be laid, just before paving operations.

The main rolled embankment for Mt. Tabor reservoirs, Portland, Ore., were built in 8-in. layers, wet when necessary and rolled with a 14-ton roller. After 6 months, settlement was ¼ in. on a fill of 52 ft. (E. R., May 6, 1911, p. 489.) An earth dam at Kettering, England, had following dimensions: top width, 12 ft.; maximum hight, 46 ft.; inner slope, 1 on 3; outer slope, 1 on 2¼ and 2¾. Puddle core was 3 to 5 ft. thick. When the structure was within 7 ft. of finished hight, the puddle wall settled and spread to amount of 7.5 ft. At the same time, upstream slope began to bulge, total motion being 2 ft. This was checked by driving a row of piles parallel to the toe of the inner slope, and about 20 ft. from it. Against this row of piles, earth was placed and carried up the old embankment. (E. R., Vol. 52, 1905, p. 365.)

Percolation Tests of Embankment Materials. To determine the relative value of various materials for the earth dikes of Ashokan reservoir, tests for percolation were conducted. The standard degree of compacting was to weights of from 125 to 130 lb. per cu. ft. To accomplish this, the material was tamped with a 4-in. circular iron tamper struck with a hammer. Water was applied through a ⅜-in. pipe in the cap by a force pump; the percolation was determined by weighing the water passing through the pipe at the bottom. Pressures were applied in increments of 5 lb., each pressure being maintained three or more 5-min. periods. In Fig. 102 are the results of some of these tests.

Nos. 3, 7, and 2 are standards, adopted as such by reason of their pronounced characteristics. No. 3 is a natural bank sand; No. 7, material of a gravelly nature; No. 2, highly impervious glacial till. While the tests do not give absolute values, by comparison with standards they are of great assistance in determining acceptability.

Tests at Titicus dam,* by measuring the hight of water in pipes set in the embankment, showed that the water-level in the upstream pipes fluctuated with that in the reservoir, while that in downstream tubes corresponded more closely with ground-water level, thereby indicating that the core wall formed a barrier to the passage of water. From these experiments, it was concluded that shape of dam, natural or artificial underdrains, springs or leaks (other than

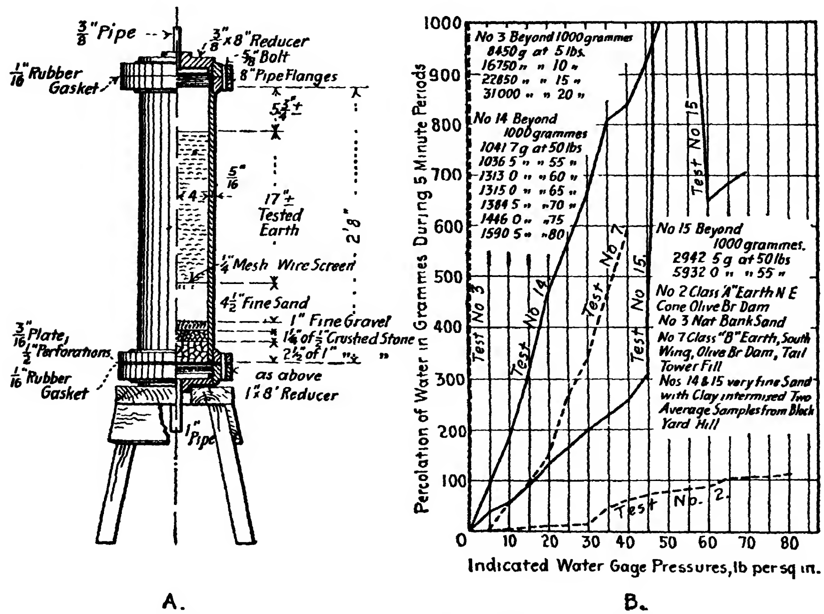


FIG. 102.—Ashokan reservoir. Percolation tests. Apparatus and diagram.

percolation) controlled amount of saturation, if not entirely, fully as much as the imperviousness of the core wall and earth embankment. The greatest lesson from the Croton dams is that the only reliable way to provide against saturation is by drainage of the downstream bank to extent that it is desirable to control saturation, and that shape of dam for a bank of uniform material has more influence on the plane of saturation than any other one factor. A core wall cannot be depended upon to prevent saturation of the toe.

Questionable Materials. Some earths which when thoroughly saturated would assume slopes flatter than the face of the dam may be used to a limited extent in dam of liberal section by mixing with more stable materials or by placing in the heart of dam with a considerable thickness of better material

* On Croton watershed. Core walls of this dam and other Croton dams tested, are of rubble masonry in Rosendale natural cement mortar.

outside. This may be the more safely done if a heavy stone layer is to be put on the face of the water slope.

Drainage. Downstream slopes should be protected from erosion by preventing concentration of surface drainage. Gutters should be made along the inner sides of berms and water thus collected taken into a pipe drain laid beneath. In a long dam, pipe drains should be laid from the berm drains at suitable intervals down the slope and the drainage led to a safe point of disposal below the toe. Even in the best built dams some seepage will occur beneath and through the embankment, consequently provision should be made for this water to drain away without causing sloughing of the downstream slope or washing away material from the toe. To this end the more pervious materials are placed in the downstream part of the dam and a blind stone drain laid under the toe with a collecting pipe drain beneath. From the latter drain a pipe with tight joints is laid to a safe point of discharge and arrangements are commonly made for observing or measuring the flow, so that if increase occurs, reasons may be promptly sought and if necessary corrected. If seepage is found slightly muddy it indicates that fine material is being washed out of the dam. This means that a water passage is being formed which is likely to enlarge, perhaps rapidly, and cause trouble or disaster. Manholes should be built on pipe drains at angles and other convenient points for inspection and cleaning. A weir or orifice with recording gage in a covered chamber is a convenient means for measuring seepage.

Foundations for earth dams should be stripped of all vegetation and top soil, or loam, and compacted by rolling or other means. From the upstream half tree roots should be grubbed out, pervious materials removed and such other steps as the site may demand taken to prevent seepage beneath the dam. As an additional precaution against seepage beneath the dam, the foundation is sometimes trenched, furrowed or stepped.

Culverts of concrete or other masonry, for the stream or other drainage during construction, can sometimes be built and the dam be constructed over them instead of leaving gaps. Cut-off walls must be built beneath and around such culverts and every precaution taken to prevent flow of water along them through the dam. Likewise they should be carefully filled wholly or partially with masonry, when the passageway is no longer needed, before water is allowed to attain much depth in reservoir.

Sequence of Construction. Construction should begin at the bottom of the valley and be carried up substantially level. If a temporary gap must be left for stream, railroad or highway, great care must be exercised when filling it to make impervious junctions. Sides of gaps ought not to be very steep and should be trimmed free of poorly compacted material just in advance of placing earth in the gap.

Construction Methods. Earth may be brought on to the dam by wagons, cars, cableways or baskets carried by men. It should be deposited systematically, spread to layers of uniform thickness by scrapers or by hand, and stones of larger diam. than the thickness of the layer after compacting removed. Even much smaller stones cannot be permitted in nests or as a large proportion of the material. If selection or mixing of earth is necessary, it can be accomplished by proper placing of loads as delivered; harrowing may sometimes

be advantageous. If the earth is not sufficiently moist as deposited, each layer must be sprinkled lightly during compacting. Excessive wetting must be avoided. Rain often causes serious delays, especially in spring and fall, as work cannot be resumed until the embankment has dried.

Compacting is commonly done with steam or horse rollers having loose rings, grooves, cleats or other devices for concentrating the loads. If the earth is brought in wagons, much of the compacting can be secured by judiciously distributing the travel. Droves of horses driven back and forth are effective. The number of passes of the roller needed depends upon its weight and several other circumstances and can best be determined by trial. In general, the upstream half of the dam (*i.e.*, the portion intended to be most impermeable) should be so compact that, after removing the looser surface material at any stage of work, a pick, crowbar or shovel cannot be driven into the bank more than an inch or two by one strong stroke. Thickness of earth layers in the most impermeable part of a dam is commonly 3, 4, 5, or 6 in. after compacting, depending upon character of materials, degree of security desired, permissible expenditure and judgment of the engineer. In remaining earth portions, layers are often of considerably greater thickness and less thoroughly compacted. Nowhere may work be so done that any serious settlement or slip can occur if the material becomes wholly or nearly saturated, as is frequently the case, due to seepage or rain or both.

Outlets from reservoirs with earth dams should not be bare pipes or conduits of any kind laid through embankment below water level; unequal settlements, even if small, may cause breaks; and water is likely to follow along outside of conduit. Geology, topography and magnitude largely determine best method. If rock is near surface or dam is on very compact earth, outlet may be in trench beneath embankment, refilled with concrete or other hydraulic masonry, with frequent cut-off collars or walls of masonry to prevent flow of water along outside of masonry. Great care must be exercised in compacting embankment to get tight contact with conduit at every point. Such outlet should have inlet gate or valve tower in reservoir near heel of dam; for large reservoirs a second gatehouse at toe of dam is usually advisable, to contain regulating or control valves. Greater security is thus afforded as well as convenience and sometimes lower total cost; one or two large conduits may connect the gatehouses and large number of delivery or distributing pipes lead from downstream house. In some situations a tunnel around end of dam is best outlet; in others outlet may be distant from dam, either tunnel or conduit in deep trench, through natural ridge. If dam has substantial masonry core-wall, inlet gatehouse may be built integral with it, on its upstream side, and have wing walls extending through upstream portion of embankment, thus bringing reservoir water for full depth against upstream face of gatehouse; or if foundation is suitable, instead of wing walls a large culvert may extend from lower part of gatehouse, beneath dam, to deep water, or there may be two or more culverts superposed, in order to draw water from different depths. For small works siphon pipe over dam may be satisfactory. Means for removing air at summit must be provided. Main requirement is to prevent flow of water outside conduit, where it may carry materials from embankment or natural walls of reservoir so as to cause a breach.

Details of Construction and Design. (1) Secure a firm, dry impermeable foundation, either by selection or drainage. Alluvial soil containing organic matter, and all porous material, should be removed. Lead springs outside the site. Drainage systems must be so designed as to prevent capillary action from soaking the embankment. (2) Tie in the embankment by durable, but elastic, impervious material. If depth to suitable foundation is great, a puddled trench will suffice. Puddle core walls are not to be carried much above the original foundations. (3) Continuity of planes between layers should always be broken. Avoid formation of cavities and lines of cleavage. (4) Make the embankment heavy enough to have a safety factor of 10 against sliding. (5) To reduce settlement, materials in the body of the dam should be as nearly homogeneous as possible. This calls for small loads and good inspection. (6) Consolidation is the most important process. (7) Use only enough water to attain consolidation. The quantity can be determined by local experiments. "Quaking areas" must be removed, and compactible materials substituted. (8) Do not rely on settlement to produce consolidation; it must be effected by mechanical means. Light rollers should be avoided as deceptive. They produce the appearance but not the fact of consolidation. (9) Avoid carrying pipes or culverts through any portion of an embankment. (10) Protect both faces of a dam against deteriorating influences, by riprap, sodding, drains, berms, etc. (11) Provide an ample wasteway in a safe situation. (Burr Bassell, Earth Dams, p. 63.)

Table 49. Pressures per Square Inch Developed by Road Rollers. Penetration = 1 In.

Makers	Number of wheels	Nominal tonnage	Weight, in lb., on wheel carrying max load	Bearing surface, sq in	Pressures, lb per sq in.
Scholl...	2	10	10,000	403	25
Scholl....	3	10	6,670	139	48
Buffalo Pitts	3	10	8,170	148	55
Longnecker.	3	10	6,670	160	42
Monarch	3	10	6,670	156	43
Kelly-Springfield.	2	10	13,330	415	32
Kelly-Springfield.	3	10	7,800	168	46
Scholl....	2	12	12,000	400	30
Scholl......	3	12	8,000	169	47
Buffalo Pitts	3	12	9,170	164	56
J. H. Strain..	3	12 5	8,400	158	53
Kelly-Springfield.	3	13	9,800	185	53
Buffalo Pitts	3	15	11,830	185	64
J. H. Strain	3	15	10,500	202	52

Weight of Rolled Earth Embankments. Average of twelve samples, rolled in layers 4 in. thick after compacting, 136.9 lb. per cu. ft. Average of eight samples, rolled in layers 6 in. thick after compacting, 138.4 lb. per cu. ft. Samples were taken from five well distributed places on dikes for Ashokan reservoir. The material was mostly clayey loam; some samples were sandy loam and others gumbo, all with varying percentages of small stones excepting that two gumbo samples contained no stones.

Cost Estimates. The volume in a typical section (Fig 103) per foot length $= 2.7\,H^2$ cu. ft. $= 0.1\,H^2$ cu. yds. H is in feet. The cost of constructing earth dams varies with the volume, and also hight independent of volume. The hight influences cost because (a) average haul is greater; (b) earthwork shrinkage is greater; (c) time is consumed in raising machinery (Prof. C. T. Johnston, Mich. Technic, 1911). However, some materials shrink little if any, and some swell slightly, *i.e.*, occupy greater space in the embankment than before excavation, even when very thoroughly compacted in bank.

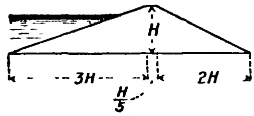


FIG. 103.

If assumed that earth can be placed in a low dam for 20 cents per cu. yd., employ the following formula for obtaining approximate cost of higher dams:

Cost per lin. ft. $= 0.1\,H^2\left(20 \text{ cents} + \frac{2H}{30}\right)$. This formula would place unit cost of dam construction 30 ft. high at 22 cents, while unit cost for dam 150 ft. high would be 30 cents. Note that H is in feet. Figs. 104 and 105 show relation of hight and cost per ft. of length.*

Damage to Earth Structures by Burrowing Animals. Of sixty-one recorded failures, most are blamed to burrowing animals, especially when other plausible reasons are lacking. Muskrats work such havoc to embankments of the Delaware and Raritan Canal that employees receive a bonus for each one killed. Animals generally attack unprotected banks, but instances are on record in India of mortar being picked out of the interstices in stone paving.

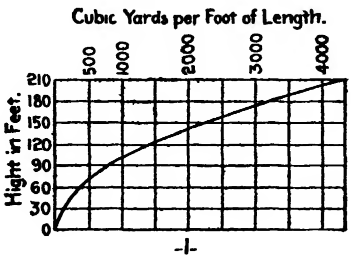


FIG. 104.

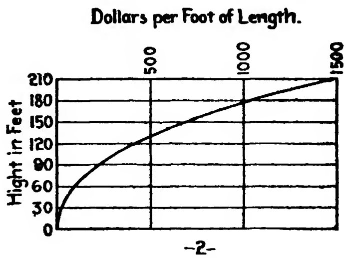


FIG. 105.

Animals found in many regions liable to injure earthworks are: Rat, mouse, weasel, common hare, rabbit, common porcupine, badger, earthworm, locust, eel, turtle, snake, crawfish, kingfisher, fox, skunk, beaver, woodchuck, mink, mole or shrew, and muskrat. The muskrat is undoubtedly the most dangerous. Animals actually known to have injured earthworks are: Muskrat, woodchuck, Indian rat, yabba, snake (in India), fiddler crab, gopher, kingfisher, mole, mollusk, eel, crayfish, mouse and beaver. The various ways of preventing inroads of burrowing animals are: (1) Resistance of hard-rolled homogeneous material; (2) a core wall of puddle, stone, concrete or timber; (3) paving on upstream or both slopes; (4) sodding downstream slope; (5) poison has been placed in the water of some navigation canals, but naturally this is not applicable to water supplies; (6) employment of men to patrol embankments. (See foot of p. 200.)

*Based on Wyoming conditions, 1904–1910.

Table 50. Dimensions of Notable Earth Dams with Puddle Cores *

Name of dam	Hight, feet	Side slopes		Top, width	Puddle wall hight, including trench	Hight of top above full reservoir
		Water	Down-stream			
Dodder, Ireland...	115	1 on 3	1 on 3	22	115	----
Stubden, Ireland. .	63	1 on 3	1 on 2	12	75	----
Peavy, Ireland. ..	40	1 on 3	1 on 2	12	40	----
Yarrow, England	87	1 on 3	1 on 2	30	173	6±
Rotten Park, England	50	1 on 3	1 on 2	20	55	----
Ulley brook, England	42	1 on 3	1 on 2	25	54	----
Vale House, England.	47	1 on 3	1 on 3	18	98	----
Llanefydd, Wales..	32	1 on 3	1 on 2	10	154	----
Vehar, India........	84	1 on 3	1 on 2.5	24	84	6 0
Pilarcitos, Cal.(*a*). .	95	1 on 3	1 on 2	24	135	5.0
San Andreas, Cal.(*b*).	93	1 on 3.5	1 on 3	24	139	5.0
Temescal, Cal.....	95	1 on 4.5	....	12	95	----
San Leandro, Cal...	125	1 on 3	1 on 2.5	28	155	----
Cuyamaca, Cal ..	40	1 on 2	1 on 2.5	15	75	6 5
Mormon Canyon, Cal	45	1 on 1.5	1 on 1.5	6	75	----

(*a*) Puddle core is 10 ft wide on top, and 30 ft wide, 92 ft. below top It is then drawn into 12 ft. width from lowest 46 ft. A concrete wall 3 ft thick extends 8 ft. into the trench and 3 ft. into the puddle. (*b*) Irregular shape, 20 ft. thick

HYDRAULIC FILL DAMS

Hydraulic-fill dams are made by excavating, transporting, sorting and depositing earthy and stony materials by flowing water; excavating, however, is occasionally done by other means. With proper materials, cost, convenience and time of construction are the only limitations to hight. Foundations must be stable and impermeable under expected reservoir pressures naturally, or must be so made. Substantially the central two-thirds of the dam should be of materials which become impervious after compacting. Slopes must be flat enough to remain stable under all conditions of saturation, during construction and subsequently. Construction must be such that after completion dam will not settle, crack or show other sign of movement. Hight and width of top are governed by same rules as for other kinds of earth dams. About one-third the bulk, equally divided between upstream and downstream sides, should be heavy, free-draining materials (small bowlders, stone fragments, gravel, etc.)

Most suitable material is an admixture of soil, clay, sand, gravel of all sizes, and rounded boulders up to the maximum sizes which can be transported with the water and grades (of sluices, ditches or pipes) available. For good results, clay should constitute 10 to 30 per cent. of the whole; it acts as a lubricant during transportation as well as rendering the dam tight. Angular stones may be used, especially if the proportion of clay is large, but they are much more liable to lodge in sluices, cause greater wear, and maximum sizes which can be transported are smaller. Pure clay is the most difficult material to use, because most unstable while saturated with the water needed to transport it and because it gives up water slowly: shrinkage is much greater, cracks are more likely to open during maturing process (drying out and settling), and

*Eng'g Contr, Jan 18, 1911, p 90.

it is more difficult to maintain slopes. Nevertheless, "wet clay under pressure will part with its water even though the mass be entirely surrounded by that liquid," and when finally consolidated, a dam of this material cannot be excelled for water-tightness and solidity. Clay makes good core, and when overlaid with sand, gravel and rock in outer portions, its surplus water is squeezed out and necessary drainage afforded. Almost any kind of clay can be used in this way, but surface clays and soils thoroughly weathered naturally are preferable to deep-seated clays in beds of such depths as to have undergone few changes by the elements. Fine sand, glacial flour, rock dust, or any finely pulverized non-plastic material is preferable to pure clay, for the reason that when settled in water it is not subject to shrinkage or further settlement, and is practically impervious if the particles are fine enough to pass a 100-mesh-per-in. sieve. Mixtures of fine sand and clay are best of all.

Most economical conditions are suitable materials at each end of dam site above level of top of dam and abundant water under sufficient head for hydraulic excavation and sluicing. But water can in some cases be pumped cheaply to give necessary pressure; materials may be excavated by other means, ground-sluiced and conveyed by flumes or pipes laid on suitable grades, and materials may be excavated from within the reservoir or other place below the dam, mixed with water and elevated by centrifugal pumps. Volume of water for a "sluicing-head" should be 5 to 20 cu. ft. per sec., although less may be used; 20 to 30 cu. ft. per sec. may be handled in one head, and is more effective proportionally than less. With sluice grades of 6 to 10 per cent. 3000 to 8000 cu. yds. of solids can be transported in 24 hrs. by a sluicing-head of 10 cu. ft. per sec. On various dams percentage of solids transported by water has varied from 5 to about 50. Angular stones not exceeding 2000 lb. and somewhat larger rounded bowlders can be carried through sluices if sufficient clay or sand is present. Sharp sand does not flow as well as rounded sand. For hydraulic excavating, pressures from 25 to 250 or 300 lb. per sq. in. may be used, and nozzles from 2 to 10 in. diam. Best form of nozzle is the hydraulic monitor or "giant" with swivel or ball joint and interior guide vanes. Foundations should be prepared as for other classes of earth dams, including cut-offs extending down into impervious material. Concrete walls, steel or wooden piling are sometimes carried up for greater or less hights into the dam, the piling being footed in concrete.

To start the dam, a levee or bank is built along toe and heel. Sluiced materials are discharged at one or more points along either side of the dam or one side only. Large stones are dropped at the point of discharge and other materials graded toward the middle, finest being deposited in the pond maintained over the central part, the sides being always kept higher. Water is drained off by standpipes carried up in the dam, a ring a few inches high being added as needed, or by troughs or perforated pipes, discharging downstream or into the reservoir as may be desired. By maintaining suitable slight grades pond water may be drained toward one end of the dam or a central point for removal. If sluiced materials are deficient in rock, gravel and coarse sand, brush is used in maintaining the edge banks or levees. Stony material as deposited is dressed by hand to the prescribed slope. Flume or pipe for con-

veying material is often supported on high wooden trestle near the center line of the dam. For large dams two may be used. Laterals are provided at convenient intervals. For small dams pipe or sluice is laid along edge of bank and raised with it, outlets being made at desired points by opening joints or by other simple means.

Stratification, *i.e.*, formation of continuous layers of pervious material through dam, is one of most important things to be guarded against. This may be controlled by even distribution of material along slopes, avoiding formation of high cones, extending beyond safe limits toward core, and by keeping pond on dam as high as possible at all times. Sand and gravel with lubricant of clay will take natural slope of between 5 and 8 per cent., but on passing into quiet pool will deposit at slope of about 1 on 1, while clay spreads through water practically level. Stratification may be corrected by systematically pushing down 1 × 12 in. wedge-shaped plank paddles to depths of about 10 ft. along lines 2 ft. apart parallel to center line of dam for a width of 20 ft. upstream from center; men work from boats or rafts and repeat operation from time to time as dam is built up. Central wooden diaphragms or partitions are also used to prevent stratification. Since during construction and maturing, hydraulic-fill dam changes from liquid mud in unstable equilibrium to solid bank, considerable settlement takes place and must be provided for. Process is sometimes very slow, and in large dam may limit rate of construction. It is good practice to test dam at all stages by permitting water to rise in reservoir to within 10 or 15 ft. of top.

Drainage of downstream portion is important in this as in other classes of earth dams. Unless approximately outer third consists of porous material, drains should be provided, extending from toe under dam, but not farther than through outer third of base. Such drains should be like filters, grading from fine to coarse materials so as to prevent water entering them from carrying fine earth out of embankment. Covered collecting well at head of each drain can be arranged to force water to rise against a slight head; each well should be surrounded with fine gravel and sand. Dams may be built with rock fill for downstream portion and hydraulic fill upstream if local conditions so dictate. To protect against rapid wear wooden flumes may have their bottoms covered with steel plates or wooden blocks set on end of grain. Most successful pipe for dam building and similar operations is one having invert of interlocking wood blocks set on end of grain, and readily removable for repairs or replacement. For helpful detailed descriptions and illustrations of many hydraulic-fill dams see "Reservoirs for Irrigation, Water-Power and Domestic Water-Supply," 1908, by J. D. Schuyler, and "Recent Practice in Hydraulic-Fill Dam Construction," by same author, in T. A. S. C. E., Vol. 58, 1907, from which above notes have largely been taken.

Wave Action.—A wind reported to have attained a velocity of nearly 100 mi. per hr. along a reservoir 5 mi. in length, drove waves over the top of a masonry dam rising 20 ft above the water surface, also above the top of heavy stone slope paving on adjacent earth dams (slope 1 on 2) with such force as to cut the slopes and do damage to the paving. This incident also showed that stones of large area, laid close together with joints tightly spalled, supported on a deep bed of cobbles and stone fragments, having large voids, were disturbed by the water held in the voids, which could not escape rapidly enough as the waves receded. Paving on such a bed should have very free vent at its surface.

Table 51. Typical Earth Dams in United States

Dimensions in feet except as indicated

Name and location	Max hight, ft.	Hight above full res level	Width on top	Slopes* (to 1) Up-stream	Slopes* (to 1) Down-stream	Berms Number Up-stream	Berms Number Down-stream	Berms Width	Core° Kind	Core° Top width	Core° Slopes to 1*	Core° Below top	Method of building	Paving	Remarks and references
Idaho Irrigation Co., Idaho.	135	---	40	3	2.5	—	—	---	...	.	.	.	-------------	..	....
San Leandro, Cal......	125	5	28	3	Varies	1	‡	---	E	30	V	125	Tamped by horses Also hydraulic process	...	1' per year of silt, E N, Sept. 11, 1902. Selected material at center
Tabeaud, Cal	123	8	20	2 5&3	2 5	1	3	10&15	P	27	25	96	6" and 8" layers Center kept 4% lower than edges	....	Settled but 2 5" with 90' of water in the reservoir.
New Croton, New York	120	24	30	2	1		2	5	R	6	045	20			
Druid Lake, Baltimore	119	5	60	4	2		.	.	P	17	01	4	Placed in basins filled with water.	P	E. N, Feb 20, 1902.
Belle Fourche, S. D.....	115	15	20	1,2 & 3	1 75&2	1	2	8	T	0 83	...	--	6" layers Piles below up-stream berm	C8"	Cut-off trench. E. R, Apr 2, 1910
Standley Lake, Cal.....	113	5	20	2 & 3	2			. .	P T	10	25	--	Core sloped to top. Sheet-piling to rock. No rolling	RC	E. R., Nov. 13, 1909.
Titicus, N Y.	110	9	30	2 4	2 5				R	5	06	4	6" layers	R_p	Paving, 18" deep.
Ashokan Dikes, N. Y..	110	20	34	2, 2 5, 2 75	2,2 75, 3	2	3	10	C	4	05	14	Petrolithic rollers†. . ..	24"	....
Temescal, Cal.........	115	5	18	3	5	0	3	---	..			..	Downstream slope built 2 5 1 and later flattened by sluicing.	S	E. N., Sept. 11, 1902.
Honey Lake, Cal . ..	96	6	20	3	2			.	P	10	off-sets	0	. ...		E N, Mar. 15, 1894.
Morris, Waterbury, Conn	100	8 75	20	2 5&3	2	0	4	8	C	2 42	05	6	6" layers	..	E N., Mar. 23, 1911.
Pilarcitos, Cal	95	5	25	2 5&3	2 5	0	0	...	P	10	0 11	3	Core-wall trench 40' deep..	S	Burr Bassell
San Andreas, Cal . ..	93	5	25	3 5	3	0	0	....	P	20	V	2	Core-wall trench 46' deep.	...	Cracks 3" wide opened by San Francisco earthquake.
Haiwee, Los Angeles..	91	10	20	2 5	2 5		..	....		.	. .	..	Washed to center........	3"	E. R, Feb 3, 1912
Forest Park, Md . .	87	5	15	3	2 5	1	0	15	P	--	--	--	Concrete bonding wall at rock	R_p	E. N., May 26, 1910.
Wachusett, N. Dike, Mass	82	15	189 17' be-low top	2	33‡ to 16‡	1	0	15	T	0 5	--	--	12" layers. Cut-off trench	S3'	Some material on 2 1 slope sloughed off. E N, Apr. 25, 1907

* Slopes are expressed as so many feet horizontal to 1 vertical, for instance, Idaho Irr Co dam has an upstream slope of 3 horizontal and one vertical.

† A petrolithic roller is a toothed roller, some 3 ft in diam, provided with a arge number of foot-like projections which penetrate several inches into the layers, and compact the subsoil They are claimed to do more thorough work than the usual cylindrical roller.

° For abbreviations, see p 216.

‡ 4 terraces where slopes change.

Table 51. Typical Earth Dams in United States.—*Continued*

Name and location	Max hight, ft	Hight above full res level	Width on top	Slopes* (to 1) Up-stream	Slopes* (to 1) Down-stream	Berms Number Up-stream	Berms Number Down-stream	Berms Width	Core Kind	Core Top width	Core Slopes to 1*	Core Below top	Method of building	Paving	Remarks and references
Cold Springs, Ore.	82	7	20	3	2								Sluicing. Cut-off trench.	R_p, 15″	20″ thick.
Johnstown, Pa.	70	— — —	10	2	1 5								Selected materials.	R	Failed by being over-topped.
Sudbury, Mass	64	7	14	2	2 &2 5	1	1	6′ & 8′	C	2′	06	4	— — — — — — — —	R_p	. . . .
Glenwild, Amsterdam, N. Y.	63 5	7	13	2	2.5	.		. .	R	2 5	25	1′ above flow line	Dam curved upstream. R = 709′.	. .	E. R., Dec. 27, 1902.
Bog Brook, N. Y....	65	8	25	2	2 5		. .	. . .					— — — — — — — —	12″	. . . . .
	25	— — —	12	2	2		1	— — —	R	6	08	— —	— — — — — — — —	12″	. . . .
Chollas Heights, Cal.	56	5	20	3	2			.	Sl	¼″	V	13	Rolled layers......	.	Steel plates end in 30″ core wall.
Cedar Grove, N. J...	55	6	18	3	2	1	1	8′	C	4	04	3 5	5″ layers. Traction engines	C	Core wall, max. 102′.
Merced reservoir, Cal	50	— — —	20	3	2			.				. .	Puddle trench Thin layers	12″	Schuyler,
Monument creek, Col	40	— — —	20	3	2								— — — — — — — —	R_p‡	E. N., Mar. 8, 1893.
Cache la Poudre, Col	38	— — —	. .	3	— — —								Scraper teams . .	R_p	Embankment settled 4′ when reservoir was filled.
Stony Brook, Cambridge, Mass.	31	5	20	1 5 &3	2	1	1	5&20	R	3	14†	3±	— — — — — — — —	R_p C	Built, 1887. E. N., June 21, 1890

Abbreviations: C = concrete, E = earth, P = puddle, R = rubble, R_p = riprap, R_p = reinforced concrete, Sl = steel, S = stone, T = timber, V = vertical.
* Slopes are expressed as so many feet horizontal to 1 vertical, for instance, Idaho Irr. Co. dam has an upstream slope of 3 horizontal and one vertical.
† Upstream face vertical.
‡ Puddle below riprap paving

CHAPTER X

WELLS[a]

PRINCIPAL METHODS AND TOOLS FOR SINKING

Outline of Methods. Deep wells are sunk by several methods, of which the following are the more important: (*a*) The "*Standard*" or *Oil-well* method is used for wells 4 in. to 12 in. diameter, through both earth and rock, to depths of about 5000 ft. Steel pipe casing is driven down to solid rock as the hole is drilled, drilling being done by a heavy plunger lifted and dropped by machinery which breaks the material into small fragments for removal by a sand bucket. (*b*) *California or Stovepipe* method is used for wells 6 to 30 in. diam. through soil. The casing consists of short sheet-iron cylinders, forced down by large hydraulic jacks and perforated in place by a special tool. Materials within the casing are excavated by a sand bucket. (*c*) *Hydraulic Rotary* method is used for comparatively shallow holes. The casing is rotated, and the water forced down through it escapes up the outside. (*d*) In the *Jetting* method the casing is sunk by driving while material inside is cut out by a high-pressure water jet through a small pipe and carried to the top of the casing by the return water. (*e*) In the *Core Drill* method a steel pipe casing is sunk to rock and then hole is drilled by a rotating bit attached to hollow rods. The bit cuts an annular hole, leaving a core of rock enclosed in the rod which is broken off and removed at intervals.

Standard Method. For very deep wells a heavy *Standard outfit* is commonly used, which may be purchased of manufacturers who make a specialty of well supplies. The principal parts are a derrick tower, engine and boiler, pitman and walking beam, drilling rope of hemp or steel, bull-wheels and wheels for cable and actuating ropes, temper screw and tools. Fig. 106 shows many of the tools.

The string of tools in deep drilling comprise rope socket, sinker bar, jars, auger stem and bit. Whether or not the complete string is used depends on conditions. The socket may have a tapered hole in which the rope is secured by knotting; rope and swivel may be riveted together, or the rope may be threaded back and forth through several holes in socket and secured by wedging. Sinkerbar is a long heavy bar, used to add weight and length and aid in keeping the hole straight; now seldom used unless a hole partly filled wtih water is being drilled. If placed between jars and bit, it adds force to blows of latter. Jars are pair of linked steel bars. When drilling in rocks in which bit is liable to stick they are necessary to "jar" drill loose, by a powerful upward blow as they are jerked violently together by stroke of the walking beam, when drill will not yield to a slow and relatively steady pull. In ordinary brittle rock the jars are now almost universally discarded, but have a very important

[a] Largely an abstract of U. S. Geological Survey, Water Supply Paper #255, 1910, by M. L. Fuller and #257, by Isaiah Bowman, 1911.

use in "fishing" for lost tools; those intended for this purpose are made longer than in ordinary drilling. Jars do not act as a maul to drive the drill into the rock; a good driller so adjusts the cable that it is impossible for the upper jar to strike the lower except when the cable is raised. All joints of a string of tools have taper screws so that only a few turns are required to fasten them together. They are screwed up tightly by heavy wrenches on which great leverage is exerted by a rachet floor circle and jack. When first screwed together tightly each joint may be marked with a cold-chisel cut across it;

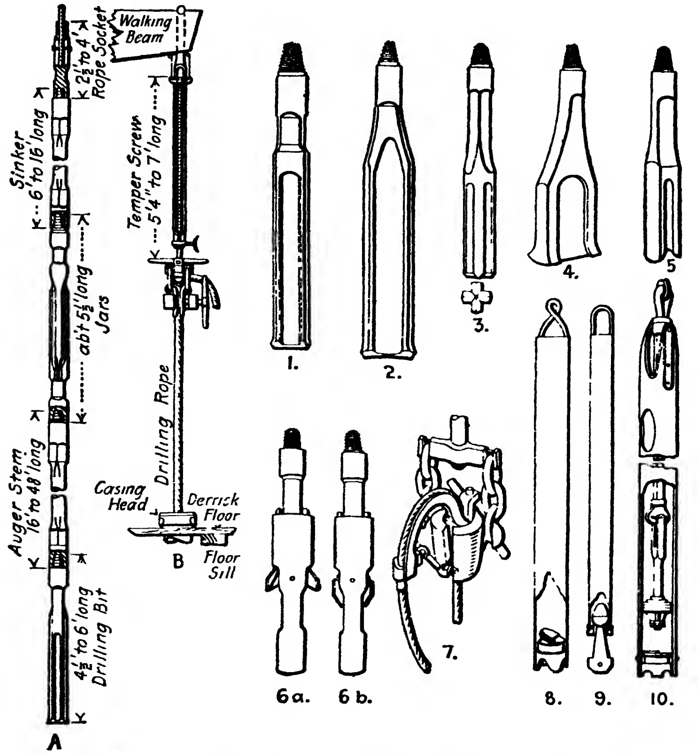


FIG. 106.—Drilling tools used with standard drilling outfit.
A. String of tools. *B*. Temper screw. 1–5. Bits. 6*a*, 6*b*. Reamers. 7. Rope clamp. 8, 9. Sand bailers. 10. Sand pump.

each time a joint is put together it is screwed up as far or a little farther. If halves of the chisel mark fall short of coinciding, sand or mud in the threads may be the cause, and, if not removed, may work out, leaving the joint loose, and cause loss of all tools below.

The temper screw, which allows the drill to be fed down, includes the frame or reins, at whose lower end is a split nut, held together by a yoke clamp. Through this nut the main screw passes, and to its lower end is fastened a handle, by which it is turned. Below the handle is a ball-bearing swivel, from which depend the clamp links, and rope clamps. The cable is gripped tightly between the rope clamps by a *C* clamp and is prevented from slipping by a "cat," made of strands of raveled rope or strips of coarse cloth, loosely plaited

or twisted to form a cord about as thick as a man's finger in the middle, and tapering toward each end. The "cat" is wound about the cable at points where the clamps are attached, and the set screw is then turned so as to bring the rope clamps firmly against it.

Rope. The best drill rope is that made of manila hemp (not common hemp nor sisal) hawser laid, that is, of three ropes of three strands each, twisted together into a single rope. In its manufacture a "nap" is formed of ends of hemp fibers, nearly all of which point toward one end of rope. The rope socket should be fastened to the end toward which fibers point, for although when the tools are attached to this end the fibers spread out and retard the downward stroke in a hole partly filled with water, they protect the rope better from being frayed by rubbing against the casing on the upstroke. *Steel-wire drilling cable* consisting of six strands of seven wires each is commonly used. The careful driller never allows tools to fall as dead weight on rock, but so adjusts the rope that tools will stretch it in reaching bottom of the hole. In drilling a deep well, the stretch of the rope is often underestimated, and it may happen that tools are falling when the walking-beam is rising, thus bringing a great strain on cable and making blows of drill ineffective. The operation is still more difficult when the hole contains several hundred feet of water, which interferes with the free upward and downward motion of cable. Steel cable has the advantage of passing comparatively freely through water. The water also reduces the shock of the steel cable by acting as a deterrent to rapid drop of tools. But a steel cable has little elasticity, and drilling by stretch of rope is hardly possible. Some drillers use 150 or 200 ft. of hemp rope between tools and the steel rope; this gives more elasticity to cable and rebound to tools.

Spudding is drilling without the walking beam—method nearly always used in sinking first 75 or 100 ft. as the string of tools is too long to be operated from the walking-beam in beginning work. Owing to short length of cable between tools and walking-beam there is little "spring" in the rope, and hole must be spudded to a sufficient depth to allow a considerable length of cable to come between; otherwise the blow of the drill will be dead, and rope likely to break. A short cable is run through the crown pulley at top of derrick, one end being attached to the bull-wheel shaft and other to the rope socket to which are usually screwed only the auger stem and spudding drill. The drill may be given an up-and-down motion in two ways: In first, rope is carried around the bull-wheel shaft in two or three turns, its end being left free. A man grasps the free end and gives a slight pull causing the coils to tighten and grip the revolving shaft; when the rope is slackened the tools fall. In the second, the drill rope is wound firmly around the bull-wheel shaft and passed through the crown pulley; a jerk line is fastened to the wrist pin of the band-wheel crank, brought inside derrick, and attached to the part of drilling cable which extends from crown pulley to bull-wheel shaft by a curved metal slide called a spudding shoe. By carefully adjusting length of jerk line each revolution of the band-wheel results in a pull on the line and its subsequent release, with a corresponding rise and fall of tools. As the hole is deepened the cable is let out by giving the bull-wheel shaft a partial revolution, and the spudding shoe

is slipped farther and farther down, for this downward sliding of the spudding shoe increases length of pull on the drilling cable. Until 200 or 300 ft. depth is reached it is often necessary to turn cable by hand, otherwise the drill may strike successive blows in the same place. The proper tension of cable can be determined only by practice. An old cable has more spring than a new one. Ordinarily the engine should be speeded up until the cable tightens slightly in advance of the stroke of the drill, so that when the drill touches bottom it will be instantly lifted with no time either to settle or stick. With hemp cable, at depth of 50 ft., when the drill tools are at the lowest point of the stroke, the bit should hang 2 or 3 in. above the bottom of the hole; at 100 ft., 4 or 5 in.; at 200 ft., 6 to 12 in., etc. To restrain loose material which would impede drilling, a conductor box of plank, circular, square or octagonal, 8 to 20 in. across, is sunk to rock, if the rock lies only a few feet below surface, the necessary excavation being done by hand; if soil is deep, spud down a hole into which a section of large iron pipe may be sunk as fast as drilling proceeds.

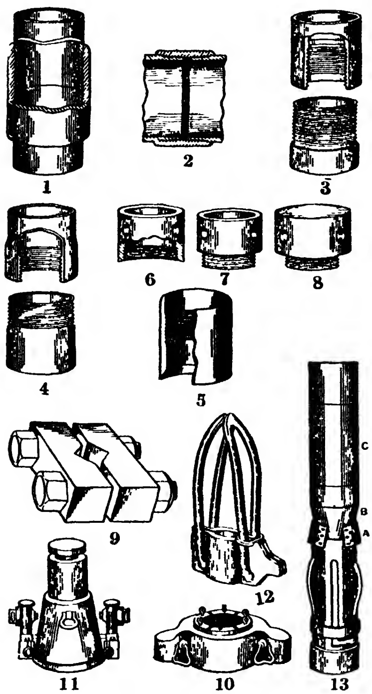


FIG. 107.—Couplings and casing attachments.

1. Sleeve coupling. 2 Tapered sleeve coupling 3, 4 Sleeve and inserted joint couplings 5 Shoe. 6, 7, 8. Drive heads. 9. Drive clamps. 10. Pipe ring. 11. Hydraulic jack. 12. Elevator 13. Gas packer (used in gas wells to isolate productive layers).

Casing and Drive Pipe. In deep drilling, where caving material may be encountered, it is customary to sink casing, or drive pipe, as fast as drilling proceeds. On bottom of lower joint there is screwed or shrunk a shoe of tempered steel, which will stand heavy driving without injury, and which gives clearance for pipe and couplings. To prevent top of pipe from being battered, a drive head is placed on it. In some places material is so loose that the tubing will follow the drill for some distance without being driven; when driving becomes necessary drive clamps are bolted to the pin square of upper end of auger stem. The machinery is then coupled as for spudding, and fifty or sixty blows a minute are delivered by driving clamp to the drive cap. The pipe may also be driven by a heavy wooden maul attached to the drill rope. In this case a solid drive head is used, and temporary guides set in the derrick.

If in drilling through a compact formation such as clay, pipe becomes lodged, owing to projection of couplings, a cap is screwed to the drive pipe, and water is forced down so as to come up outside the pipe and loosen material. Endeavor to keep the hole a few feet ahead of the casing, for then the casing is let down straight; if pipe is driven ahead of the drill, casing may be deflected. This can be done only where the material will stand up for some distance with-

out support. Quicksand will flow around the bit and even rise in the pipe; in this material the pipe must be kept ahead of the drill; where beds 50 or 60 ft. thick are encountered, some of the greatest difficulties known to well engineers have to be overcome. When long strings of casing or drive pipe are to be "pulled," a pipe ring is placed over the upper end and subjected to upward pressure by special hydraulic lifting-jacks. With short or easily pulled lengths, wrought-iron elevators clamped to the upper end are caught by a hook suspended from the derrick, and tugged upward by means of pulley sheaves. Several machines for pulling casing are on the market.

Portable Drilling Rig. Several modified forms of the standard outfit, called portable rigs, are mounted on wheels. Horses are commonly employed, but in some localities a traction engine supplies power for operation and hauls it when moved. Total weight of a portable rig is about 15,000 lb.; lighter outfits may weigh only 5000 lb., and heavier, 20,000 or more. Beyond 1500 ft., the stroke of most portable engines becomes so short as to make them less efficient than standard outfit.

The pole-tool or wedge-rod method of drilling differs from the standard chiefly in using wooden rods instead of cable. It is still used in the area between Illinois and Montana and north of Kansas and Colorado, where great depth of wells and the large amount of water encountered make wood rods advantageous. If a portion of the hole is drilled dry, a cable is attached and drilling is conducted as with standard outfit. Experience has shown that under ordinary conditions expense of casing and of increased labor in drawing or perforating it, when drilling water wells with standard outfit, exceeds cost of drilling with pole-tool outfit. If the hole is full of water and its size is decreased to 3 or 4 in., drilling is impeded by column of water that rises and falls with each stroke; the diam. of hole, therefore, cannot be reduced below the size which gives ample clearance between rods and well wall. In the pole-tool method, the drill falls on rock as dead weight; this reduces speed. In quicksand pole-tool is almost worthless because of great time in withdrawing tools and rapid inflow of sand; it must be supplemented by rotary appliances.

The self-cleaning or hollow-rod method includes the essential features of the percussion methods, but differs in combining in one operation the breaking up and removal of material. Tools consist of a string of pipe with screwed couplings, usually of special manufacture to insure strength and durability, with water swivel at the upper end and drill bit at lower end. The water swivel is essentially a swiveled gooseneck that allows water and drillings to be discharged in a constant direction and still permits the drill pipe to be rotated. In usual outfits 1½ or 2-in. pipe and a drill 2½ to 4 in. across are used, though hole may be enlarged by use of an expansion bit. The drill bit has one or more holes at its upper end and a flap valve that allows water and drillings to enter the pipe and be lifted automatically to the surface. Blind valves are also usually placed at intervals within the pipe to relieve lower valve of weight of entire column of water and drillings. The self-cleaning outfit is admirably adapted to sinking wells of small diam. in sand, clay, shale, soft limestone and other easily penetrated materials. The "Ohio" machine is a self-cleaning outfit fitted with a special operating device; rods are gripped by jaws and lifted by power transmitted through a crank and pitman. At

upper end of stroke levers attached to the jaws are tripped, the tools are released and fall freely, and are caught up again by the jaws on the rebound.

California or Stovepipe Method. For unconsolidated alluvial deposits a method is used which on account of its origin, is called California; or is called stovepipe, from nature of its casing; also, mud-scow method, because of the instrument used for drilling and bailing. The rig consists of a horizontal rectangular frame, to which is hinged a derrick about 34 ft. high, held vertical by two diagonal braces from frame. The walking beam is carried at top of derrick. Bottom of the derrick is hinged to a saddle piece resting on two coiled car springs, which in turn rest on a heavy crosspiece bolted between the sides of frame. The hoisting reel and other machinery are placed on the frame back of the derrick, and farther back are the engines and the hydraulic pumps. When steam is used, the boiler is separate. When the machine is to be moved, derrick is swung back on hinges with top resting on an upright at rear end of rig, jackscrews are placed under the sills and the machine is raised sufficiently to allow wheels to be slipped on to axles bolted to the sills. In drilling, the walking-beam is given a reciprocating motion by a wooden rod attached to its rear end and to the driving machinery. Tools are suspended from its forward end by a cable, usually ½-in. wire rope. The sand bucket has a flap valve and is connected to the tools by a knuckle joint that permits easy dumping, and carries a cutting shoe similar to that on bottom of a string of casing. For clay this shoe may have a straight chisel-like bit extending as a diam. across it. Casing is made of lap-riveted or welded cylinders of sheet iron or steel, usually No. 10 to 14 gage, 24 in. long and 6 to 16, and sometimes 30, in. in diam. Metal may be taken to the field flat and riveted there, but usually the riveting is done at shops and ends turned square and true. Two sizes are used, one of which just slips within the other, so that the joints of one may be adjusted to fall midway between joints of the other. Sections are added one at a time as sinking proceeds, each 2-ft. section adding 1 ft.; outer and inner sections are united simply by denting with a pick. Casing is water-tight, but for water wells it is not necessary that it should be. Casing may be started from a properly recessed drive shoe, but it is easier to begin a well straight and keep it plumb by using a starter, 12 to 20 ft. long, of sections of stovepipe casing, often three thick-

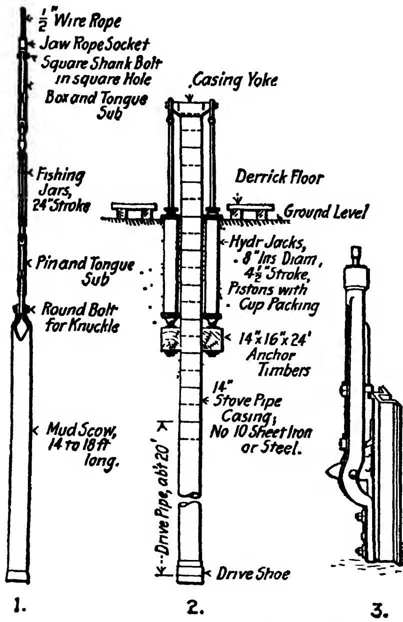


FIG. 108.—Parts of California outfit.
1. String of tools 2 Casing and jacks 3 Perforator

nesses, riveted together, or a section of heavy lap-weld casing. At lower end is an annular steel drive shoe. Casing is usually sunk by two or more hydraulic jacks buried in the ground. In small shallow wells casing is sometimes forced down by two steel I-beams or railroad rails, arranged as powerful levers.

Water is necessary for drilling and must be supplied until struck. The sand bucket must not be filled to overflowing, or the excavated material may spill over and stone and gravel become jammed. Boulders are worked to one side, or broken by a drill substituted for the sand bucket. In quicksand or caving ground it is necessary to keep the water-level about the same inside casing as on outside, also to keep casing even with, or ahead of, excavation. In very soft ground a hole larger than the casing is apt to form, and falling ground from top of such a cavity is liable to crush the casing.

Perforation of Casing. A record of material encountered is kept, and after well has been sunk to required depth a cutting knife is lowered, and vertical slits cut in casing at selected water-bearing strata. One perforator is shown; another has a revolving cutter that punches five holes at each revolution of the wheel. Vertical slits are of a form and size that do not clog readily. For best results in fine material, a natural strainer is formed about casing by removing the finest material adjacent to pipe by pumping the well as low as possible for several days. Sand bucket gives better knowledge of strata penetrated than "wash samples." The yield ranges from 0.05 to 5 mgd. according to conditions. Seldom or perhaps never is it necessary to re-perforate a well casing; the yield of an old well may often be increased by using compressed air, or by heavy pumping, loosening surrounding sand and gravel.

Durability of Casing. Stovepipe casing is often not as durable as others. Experience in southern California has been that a stovepipe casing, well below surface of saturation, in water containing little oxygen or carbon dioxide, suffers no appreciable deterioration. Wrought-iron casings have been taken from the ground as good as new after 20 yrs.' service. At the ground line corrosion is the greatest, and light casing does not last more than 10 or 15 yrs.

Advantages of stovepipe casing are: (1) Smoothness offers no resistance to the tools inside, and minimum to the material without. (2) If properly made, casings are not weak at joint like screw-pipe, but uniformly strong. (3) Great flexibility. (4) The short lengths of casing permit the well to be sunk by jacks of limited working range. (5) Uniform pipe allows perforations at any points that the material indicates. (6) Cheapness. (7) Hydraulic efficiency; great numbers of small holes allow for high yield with low velocity outside. (8) The absence of perforations in any part when first put down permits easy use of sand pump, and the penetration of quicksand, etc. (9) Deep wells with much screen may be drawn upon heavily with little loss of head. (10) The perforations are the best possible for the delivery of water and avoidance of clogging. (11) The large size of casing permits a well to be put down in bowlder wash where a common well could not be driven. (12) Uniform pressure by the hydraulic jacks is safe, convenient and speedy compared with driving of casing by a weight. (13) Good progress is made in

material that would be considered in many places impossible to drive in by other methods.

Los Angeles Stovepipe Wells. On work at Los Angeles, a pair of 6-in. hydraulic jacks exerting 50-ton pressure with a water pressure of 2000 lb., was used for first 300 ft.; also a pair of 8-in. jacks giving 100 tons under the same water pressure, was used. Wells, 29 yrs. old, are apparently still in good condition; never known to fail below the permanent water surface although they sometimes rust at unsubmerged points below the ground.

Hydraulic rotary method has been used many years for sinking shallow wells in fine-textured, unconsolidated materials, but attained prominence in deep-well drilling in 1901. A derrick like that of a standard rig is used, but machinery and tools are unlike those of a percussion outfit. The principal parts are a revolving table or whirler, two hydraulic pumps, boiler, engine and a line shaft. The table is a heavy rotating device, on tool-steel rollers, that grips the casing firmly, revolves it, and yet allows it to be lowered gradually. The table is revolved by bevel gearing connected with the line shaft and controlled by a clutch and lever. Pumps are capable of developing 125 to 175 lb. per sq. in. As the water is very muddy, pumps are made with but 6 to 10 valves; the valves also fit more loosely than in ordinary pumps, so as to reduce the abrasion. Only one pump at a time is in use, but to permit packing of valves and provide for emergencies two pumps are always installed. The pump is connected with top of casing through a hose and water swivel similar to those in the self-cleaning outfit, but larger, so as to allow continuous pumping into the casing while the latter is being rotated. Thin mud or slush plays an important part in drilling, and a slush pit, an essential accessory, is usually dug near derrick, on same side as the pumps, about 40 ft. long, 15 ft. wide, and 3 or 4 ft. deep. A ditch where sand may settle out of the mud is cut from the well circuitously to the slush pit, from which hose or pipes lead to the pumps.

Drilling is accomplished by rotating entire string of casing with toothed cutting shoe on the lower end. Only one casing, that being revolved, is used, for as muddy water escapes upward it puddles the side of the well so that material stands alone. If clay is encountered, water used in drilling is kept as clear as possible and not drawn from the slush pit, for the clearer the water when introduced the greater its capacity to uphold and move particles of earth, and clay is sufficiently compact to make a wall that will not cave. In penetrating sand and gravel, clay often has to be added to the slush pit, so as to make a thin mud that will plaster up these beds and prevent escape of drill water. Many clays are so compact and dry as to resist the action of water, and if the casing is fed too rapidly a core forms reducing size of opening through which the water must pass, and correspondingly increasing the pressure exerted by pumps; this may be obviated by fastening across end of shoe a bar that will cut the core. Pumps must be kept going constantly, otherwise drillings will settle and "freeze" pipes fast. One size of casing is carried down until friction renders its turning difficult, or it becomes stuck fast, from some other cause. Then a smaller size is used until it in turn becomes fast.

In penetrating firm material it is sometimes necessary to employ a rotary drill bit instead of shoe; two styles are in general use: diamond-shaped and fishtail. The diamond-shaped is usually first employed, and the fishtail

afterward for enlarging. These bits are used on the smaller casing and slip down inside the larger casing. In hard rock chilled shot or other abrasive may be used as in the shot method. Many water wells sunk by the hydraulic rotary method are difficult to screen because of depth at which the operation is conducted and fineness of material. One method is to puddle the wall of well at water-bearing layer, set the screen, and draw casing to top of screen. By pumping heavily for a few hours, puddled sands are partly reopened, but the method has the defect of leaving water-bearing layers more or less clogged by fine material. A more difficult but better and more common way is to sink the screen below the casing by forcing a hole down by a jet of water, the wash-pipe being run ahead. A packer or lead seal is then inserted at the point where top of screen joins well casing to prevent materials from rising over top of screen and filling it.

Jetting Method. In the jetting method, material is both loosened and carried to the surface by water under pressure. The principal parts of

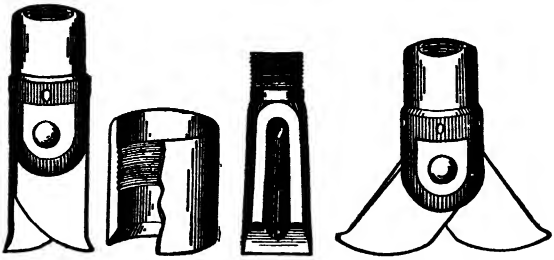


FIG. 109.—Jetting outfit.
1. Paddy (or expansion) drill closed 2 Drive shoe 3. Common jetting drill 4 Paddy drill in widest position.

the outfit are force pump and water swivel, drill pipe, nozzle or drill bit, casing and drive weight. Water is led into the well through a pipe of relatively small diam. and forced downward through the drill bit against the bottom of the hole. The stream loosens material and the finer portion is carried upward and out of the hole. The drill pipe is turned slowly to insure a straight hole. Casing is usually sunk as fast as drilling proceeds. In softer materials, by using a paddy or expansion drill (which opens when it strikes bottom: Fig. 109, 1 shows closed position, Fig. 109, 4, open) hole may be made somewhat larger than the casing, which may be lowered a considerable distance by its own weight. Ordinarily, however, a drive weight is necessary to force it down. As a rule one size of casing may be employed for entire depth. It is usually difficult to drive a single casing beyond 500 or 600 ft., and if well is much deeper, a smaller size must be used. In fine-textured, clayey or loamy material, hole may be jetted down to full depth and casing inserted afterward.

The jetting method is much employed for putting down wells in the Atlantic Coastal Plain and in some of the valleys of the arid West deeply filled with alluvium. In Coachella Valley, southeastern California, the method has been successfully used for flowing artesian wells. Wells were usually 400 to 500 ft. deep, 4 in. in diam.; not uncommonly a well may be sunk, cased and cleaned in two days.

Core drills are little used for sinking wells, though tried from time to time; diamond drills have been employed to some extent in South Africa for deep-water wells. The core drill principle is, however, occasionally employed in connection with more common well-drilling outfits. All rotary core drills are portable, can be taken apart for transportation on pack animals and used where more cumbersome outfits are debarred. Nearly any power can be used —electricity, compressed air, steam, gasoline, horse or hand.

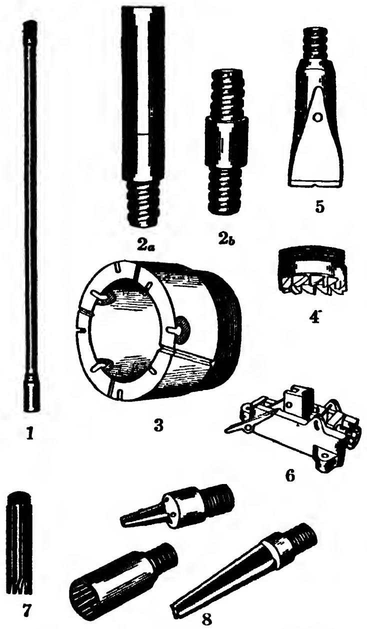


FIG. 110.—Parts of core-drilling outfit.

1. Davis calyx drill rod with coupling attached. 2*a*, 2*b* End of drill rod and rod coupling 3. Bit to be set with diamonds 4. Toothed cutter bit. 5 Chopping bit 6. Safety clamp (prevents loss of drill rods when hoisting out). 7 Toothed collar. 8. Coupling recovery taps

Diamond Drills. Two kinds of diamonds, known as carbons and borts, are used. Carbons are opaque, irregularly shaped nodules, black on outside and shades of gray on broken surfaces; they have no cleavage planes, and it is this quality, together with their hardness, which especially fits them for drilling in hard rock. Borts are semitransparent, similar in appearance to the rough brilliant but differing in crystallization; they are usually nearly spherical, as hard as carbon but not as tough, and have a cleavage plane so that in hard rock they may break; whereas carbons wear, but seldom break. Carbons come mainly from Brazil; borts from Brazil and South Africa. Borts are much cheaper. Diamonds must be selected with special attention to uniformity of size and weight, as irregularity will disturb the balance of stones and necessitate frequent resetting. They usually require resetting after 8 to 12 hrs. work. The size of diamonds ranges from about 1 to 4 carats, according to size of bit. Stones of approximately cubical shape are best, as they are stronger and furnish better cutting faces. In starting, the first requirement is to get down to rock. If the soil is thin, a pit is dug, drive pipe inserted, and tight joint made by chiseling a seat in the rock, driving the pipe down and calking it firmly. If soil is more than 10 or 12 ft., it is cheaper to drive the pipe to rock. Where drive pipe cannot be seated with sufficient firmness to keep out surface water, a hole is drilled inside with a chopping bit, and a string of casing put down to a depth sufficient to exclude water.

Calyx Drill. The hoisting and driving machinery of the calyx drill is similar to that of the diamond drill, and feed water is supplied through a swivel and hollow drill rods. The bit is of hardened tool steel and consists of a toothed collar, somewhat like the cutting shoe of the hydraulic rotary outfit, but having a longer barrel and teeth, the teeth being so set as to provide clearance to the core and to the bit and rods. Above the core barrel a

cylindrical chamber, or calyx, open at the top, encircles the drill rods. In it coarser rock fragments torn off by the bit are caught as dropped by the upward water current when its velocity decreases. They are removed when the rods are hoisted, and furnish a second record, in inverted order, of material penetrated, of especial value in material too soft to yield a core.

Chilled Shot Method. Experiments in drilling by loose abrasives poured down the drill hole led to adoption of chilled steel shot, such as is used in sawing stone. Other parts of the shot outfit are similar to the diamond drill, but cutting is accomplished by revolving an iron or steel tube on the shot. A slot a few inches long and half inch wide cut into the lower end of the tube or bit allows shot to reach the cutting surface more readily and be more evenly distributed. Distribution is also aided by slightly beveling the edges of tube so that shot may get under it. Under weight of the drill rods the shot bites into the rock and chips or wears off small pieces, which are brought to the surface by the water current.

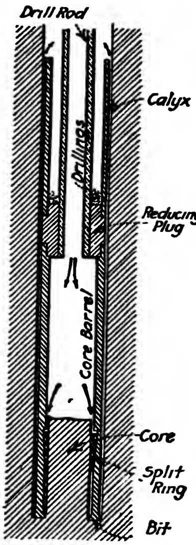


FIG. 111. Calyx core barrel.

Precautions. In sinking important wells: (1) Accurate log should be kept, so that depth and character of water-bearing formations may be known. (2) Every water-bearing layer should be carefully examined as to thickness and quality of water. (3) Head of water of each water-bearing layer and its relation to other water encountered should be accurately determined, so that, if necessary, contamination may be prevented by using packers and separate pipes for each water horizon. (4) Casing should be intact when well is completed, and should be kept so in order that it may fulfill its duty in shutting out undesirable water. Its condition should be determined from time to time by suitable experiments. (5) Possible effects of defective casing should always be considered in interpreting a change in head or quality of water. (6) To exclude surface water tie a leather "seed bag" filled with flaxseed firmly around the pump tubing and let down to proper depth. In a few hours the seeds swell and fill space between tubing and wall of well.

Increasing Supply. Explosives are used to some extent for increasing supply and providing a reservoir for the water. If water is drawn from rock, fissuring the rock will increase the area from which delivery is made. In a limestone region, where the underground water, like the surface water, runs in more or less definite channels, instead of percolating slowly as a broad or thick sheet, torpedoing will almost surely increase the number of contributory veins. The steam jet is sometimes used in unconsolidated deposits. Steam is forced down a small pipe inside of a larger one, and coming into contact with water at the bottom turns it quickly into steam, the resulting explosion loosening the material or making a pocket about the bottom of the pipe. Where the materials are dense and clayey, action of the jet may considerably increase the influx of water; in more porous deposits, it has less effect.

MINOR METHODS OF WELL SINKING

Boring with Auger. In alluvial deposits along streams and in other unconsolidated deposits, wells 2 or 3 in. diam. are in some parts of the country bored to ground-water level with a hand auger made by welding a carpenter's auger to a rod or pipe. Auger works more efficiently if the centering point is cut off and lips are shaped as in Fig. 112(3). Another much-used auger is formed by making a spiral coil of tire iron, shaping a cutting bit on lower end, and welding or riveting to a rod or pipe. As the auger is heavy when loaded, a windlass or small derrick may be used in lifting. In boring through dry sand or other loose deposits a little water should be poured into the hole to cause the material to cling to the auger. Casing is not usually required until the auger reaches saturated sands; it may be driven by a wooden maul or ram. Mud and water-saturated sand may be bailed out with a sand bucket. Hard layers

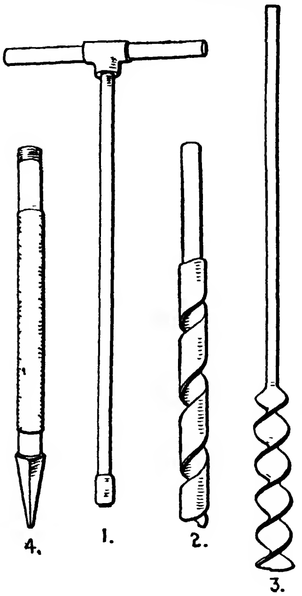


FIG. 112.
1 Small earth auger 2, 3 Earth auger bits. 4 Drive point and screen.

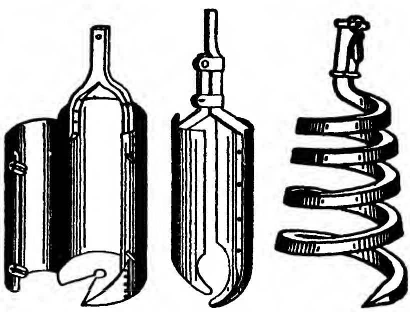


FIG. 113.—Tools for auger borings.
1, 2 Ordinary clay auger, (3 to 36 in diam) 3 Special auger for penetrating small bowlders or soft rock

may be penetrated by a drill similar to that used with percussion outfits. Where the ground holds together well an Arkansas clay auger is employed for sinking wells to depth of several hundred feet, especially in lower Mississippi valley. This auger is 15 ft. long, and consists of a cast-steel barrel 4 ft. long, which resembles a 3-in. pipe sawed vertically in half. This is fastened by a flat piece of iron to a second auger barrel, 1½ to 2 ft. long; above this is a second piece of flat iron, square at the top, and cut with threads for fastening to wooden poles. At the bottom of the auger barrel, on the right side, is riveted a steel cutting edge, commonly called the cutting bit, projecting inward 1½ in. On opposite side, and slightly above, is the "auger lip," which helps to hold the dirt in the auger barrel, when the tools are lifted. The auger is fastened to a 10-ft. auger pole, and this to the regulation 26-ft. pole. The tools are turned with a clamp, and when the bit begins to choke, the tools are lifted and dropped by means of a windlass. This operation jumps the dirt in the bit, and so frees the lower end; it is termed, "making a slip." If the clay is very dry a little water is added, and with very sticky Cretaceous clay

this process can be continued until the whole length of the auger is filled. This 15 ft. of mud represents about 10 ft. in depth. Usually the auger is filled for only about 10 ft., representing 7 ft. depth, before lifting the tools. When rock is encountered, a bar drill is used, sometimes attached directly to the wooden poles and sometimes to iron poles. When sandy layers are encountered, which will not hold in the auger, a sand pump is used, or enough clay is dumped into the hole to make the sand stick together. Where wells of larger diam. are desired, the hole is enlarged with a reamer.*

Punching. In a few localities where material is very clayey, wells are sunk by a punch. This method, used in Arkansas and Louisiana, employes a cylinder of steel or iron 1 to 2 ft. long, split along one side, and slightly spread. The lower portion, very slightly expanded, sharpened, and tempered into a cutting edge, is attached to a rope or wooden poles and lifted and dropped in the hole by means of a rope given a few turns around a windlass or drum. Material is forced up into the bit, slightly springs it, and so is held. When working in very dry clay, water is sometimes added. Thin sand layers are passed by throwing clay into the well and mixing it with the sand until the bit will take it up.*

Driving Pipe. Where the water level is within the suction limit of about 25 ft. below the surface and water-bearing beds are unconsolidated, small supplies may be cheaply obtained by driving a strainer and drive point fastened to a piece of pipe. When desirable water is reached, the well is pumped rapidly to free the strainer from particles of sand and clay. The coarser material adjacent to the strainer is thus washed free and forms a natural filter. The driven well is used to a large extent in the Atlantic and Gulf coastal plains, and is becoming common in the Mississippi valley and on the prairies. British Army officers call it the Abyssinian well, as it was first used extensively during the Abyssinian campaign of 1895. Clay and sand often cling to the screen and render it worthless. To obviate this, pipe may be driven to the required depth with only a drive point and then pulled up a distance equal to length of screen. The drive point, which fits loosely on the pipe, remains behind and the screen is lowered upon it. This style of screen can be removed and cleaned if it becomes clogged, but can be successfully used only when depth to water-bearing layer is known, since a test for water cannot be made until the pipe is raised. A modification consists of a screen attached to a drive point and so arranged that the drive pipe slips down over the screen and rests on the point while being driven. In testing for water the pipe is withdrawn a foot or two, and a pitcher pump is screwed to its upper end. If water is not procured, the pipe is driven again. The only drawback to this is that pebbles may lodge against the screen when the pipe is withdrawn, and when it is driven down again, may tear the screen. If the water contains much iron, a thick crust may form on the pipe and screen and reduce the inflow of water or even shut it off entirely. Incrustations may be loosened and broken up into fine particles that may be pumped out by pulling the pipe a few inches, by driving it down a short distance, or even by rapping the pipe sharply. While cleaning a new well, pumps should not be stopped until sand is no longer discharged, otherwise

* A. C Veatch, "Geology and Underground Water Resources of northern Louisiana and southern Arkansas." Prof. Paper, U. S. Geol. Survey, No. 46, 1906, p 95.

sand will settle about the valves of the pump and make it impossible to start again without drawing the pump rods and valves, cleaning them and resetting. If the well is sunk into unconsolidated materials, a screen must be used which will permit water but not sand in serious quantities to enter.

Objections to Driven Wells. Driven wells, as a continuous source of supply, are open to the following objections: (1) The extreme difficulty of determining the tributary area prevents a satisfactory estimate of yield and for the same reason it is difficult to remedy pollution, or trace its sources. (2) Doubt as to the permanency of the supply and possible deterioration. (3) Legal complications arising from the effect on adjacent catchment areas. (Municipalities are liable for damages where the water-table has been lowered by diverting water for public use.) (4) Infiltration of sea water, unless the wells are most carefully managed, is imminent near the ocean.

Large Dug Wells. A well at Greenfield, Mass., described by G. F. Merrill, in J. N. E. W. W. A., 1915, is more or less typical. This well is 40 ft. diam., 30 ft. deep, 25 ft. of which is below ground-water level. Excavation was made to water level, then forms were built and a reinforced concrete wall, 24 in. thick at top and 30 in. at bottom (battered), placed to hight of 10 ft., 5 ft. projecting above ground. Wall was sunk by digging beneath it. As the wall settled, concrete was added to its top. The bottom of the wall was bevelled inside to serve as a cutting edge. As the concrete was placed, a large number of 2½-in. tile pipes were placed in it to permit free passage of water through the wall. A 6-in. centrifugal pump, belt-connected to an electric motor, was mounted on brackets bolted to the inside of the wall, during construction. Since the pump sank with the wall it was not necessary to lower the suction pipe. The discharge pipe was so constructed that it would turn on an elbow outside the wall as the well sank, without straining the connections. There is a reinforced concrete dome over the well, with a manhole over the suction pipe. The dome is covered with 2 ft. of earth. The well is situated in a bed of coarse sand and gravel close to the Green river, and was pumped during 1914 at rates of 1.2 to 2 mgd.

OVERCOMING DIFFICULTIES IN WELL SINKING

Locating Lost Tools. The first step in recovering a lost tool is to learn shape of its upper end and its position in well. This knowledge may be obtained by lowering over the tool a sheet-iron vessel containing soap or other soft material, in which an impression is easily made. If caught in the well above water, or in a dry hole, the position may be determined by reflecting light into the well from a mirror. A photographic device, invented by Mr. Loran, a Baku engineer, consists of a stereoscopic apparatus, which is lowered to a point near the lost tool, light for the negative being furnished by an electric current carried by wires arranged in the camera. Some good photographs made in this way are given in A. Beeby Thompson's book, "The Oil Fields of Russia," (Van Nostrand, 1904), showing the exact shape and position of fallen tools. The drill is seldom left in the well over night on account of danger from sand or of malicious cutting of the rope, or, in a rock well, of having a bowlder fall and become jammed between drill and well wall.

Bowlders. If a bowlder is especially hard, it may be blown to pieces by dynamite or rock powder tamped with a bushel or two of dry sand or clay. This may split the bowlder so that the casing will pass down between the broken parts, or may break it into pieces so small that they can be further reduced by the drill and removed by the bailer. Casing should be drawn 3 or 4 ft. above the charge.

Running Muds and Clays. Mud produced from some shales hardens quickly when exposed to air. The hole must be cased and drilling must be

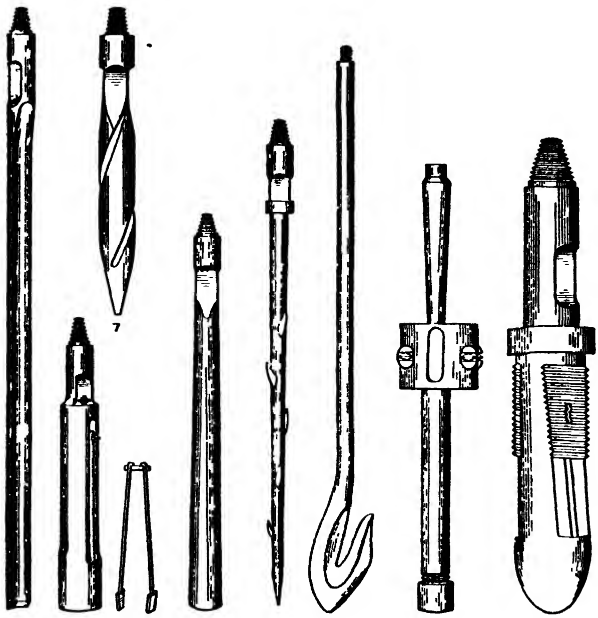


FIG. 114.—Fishing tools used with standard drilling outfit.

1. Spudding spear (prods loose a jammed drill rod). 2. Slip socket. (Teeth on slip project upward to grasp tools) 3. Horn socket (slit on opposite sides to grasp tools near surface when jammed over them). 4. Rope spear. 5. Rope knife. 6. Casing cutter. 7. Pipe swedge (straightens casing). 8. Slip socket for inside of pipe.

pushed so rapidly that the mud will not have time to solidify. The drill must be freed from this mud and withdrawn by slowly working it up and down so as to gain on the upstrokes, and the mud may be removed by small buckets or augers. If this method fails, $1\frac{1}{2}$- or 2-in. pipes may be lowered and the hardened mud and sand flushed out by a powerful water jet.

With only a small quantity of water, clay will "crawl" and relieve pressure by squeezing through very small openings in threads or sheets. Slow but forcible movement of plastic clay into a drill hole may fill the hole during a single night, when drilling is suspended; next morning the drill will strike this soft plug and ram it down until compression of the air below prevents its further movement. Drill may pound on this cushion for days or weeks without progress while clay slowly accumulates in the hole. This difficulty may be overcome by casing off the clay before it forms a plug or by jetting through the

plug. Plastic clays are encountered in South Dakota and Atlantic coastal plain, but most glacial clays are so sandy that they yield readily to the drill even if the well becomes clogged.

Quicksand. In the coastal-plain from Cape Cod westward and southward along the border of the continent the most serious difficulty is caused by quicksand, as a rule interstratified with coarser sand and clay. Quicksand comes into the hole and must be bailed out in large quantities before the casing can be driven farther and drilling continued. Under ordinary conditions quicksand will not yield its contained water, and therefore if it has a tendency to rise in the pipe, the difficulty can seldom be obviated by pumping alone. Pockets or lenses of clay or coarse sand in quicksand may cause the driller to think he has passed through the quicksand. Coarse sand, such as "bar" sand, will not rise if the velocity of the water through it is less than about 2½ ft. per sec. Drive pipe shuts off water and quicksand above a pocket of coarse sand or clay, but as soon as the drill penetrates the pocket quicksand flows in and may rise to top of deposit. If the bed is 20 ft. or more thick, the pipe cannot be driven through it on account of the resistance of the compact sand; and if the water in it is under great head, so as to force the sand up to or above the point at which the bed was struck, further progress may be almost impossible. In some wells quicksand has risen 100 ft. above the depth at which it was struck. If the hole is not kept full of water, pressure exerted by quicksand on well casing may be great. Experiments have shown that quicksand partially saturated with water exerts a lateral pressure equal to half its vertical pressure. Beyond point of saturation pressure is hydrostatic. If quicksand is only a few feet thick it may be penetrated by bailing and then driving the casing; this pipe is driven as far as possible into the bed without bailing, and quicksand may occasionally be passed through at one drive. A thin bed of quicksand near the surface may be shut off by sheet piling. Difficulty may be partly overcome by filling the bottom of the well with mortar or Portland cement, which sinks through the quicksand and sets. Stones, clay, and asphalt have also been dropped into the hole to restrain quicksand, with some success. Some drillers maintain that quicksand can always be penetrated by keeping hole full of water. If working in sand of fine texture, draw the drill at night, as otherwise sand may creep up around the drill and "set" almost as hard as rock.

Deflection of Drill-hole. In beginning a well care must be exercised to make the hole plumb; otherwise drilling to depth is difficult, if not impossible, because of friction of the tools caused by increasing deflection. To keep the hole straight the driller may lengthen tools to 60 or 80 ft., chiefly by using a long auger stem.

HYDRAULICS OF WELLS

Size.* Inside diam. of well pipe is determined to smaller degree by quantity of water to be raised than by consideration that in sandy soil speed with which water enters pipe must be below a certain value to avoid clogging. If H is hight of perforated pipe in zone of underground water, V permissible velocity

* König, Wasserleitungen und Wasserwerke, 1907

with which water may enter pipe, D inside diam. of pipe, $Q = DHV\pi \div C_p$, and $D = C_pQ \div HV\pi$, where C_p is coefficient of permeability. V is to be determined by pumping experiments; if $V = 0.0033$ ft. per sec. and $C_p = 4.0$, $D = Q \div 0.00259H$. If $H = 6.5$ ft. and $Q = 0.014$ cu. ft. per sec., $D = 0.83$ ft. If a suction pipe with external diam. D_1 is inserted in well, $D = 1.2D_1$, and $D_1 = 0.833D$.

Arrangement and Spacing.* Several wells may be arranged in a straight row perpendicular to flow of underground water, on right and left of line of most rapid drop of water, wells being so spaced that full utilization of underground stream is assured; or in two or three rows with greater distances between wells, or in a circle.

In a single row the distance between wells equals $\frac{3}{4}D_c$, where D_c is diam. of circular area from which water for each well is collected. Length of area from which water is collected is, therefore, $L = (n - 1) \times 0.75D_c + D_c$, or approximately $0.75\ D_cn$, where n is number of wells in one row. The width of this collecting area is D_c; hence area $A = 0.75nD_c^2$; for $n = 10$ and $D_c = 164$ ft., $A = 202{,}000$ sq. ft. For double-row arrangement (wells staggered) distance between wells longitudinally is $1.5D_c$, and in cross direction $1.25D_c$, and in this case length of area from which underground water is collected is $L = 1.5nD_c$. Width of area is here $2.25D_c$, and therefore area of whole collecting district is $A = 3.375nD_c^2$, and if $n = 5$ and $D_c = 164$ ft., $A = 454{,}000$ sq. ft., that is, more than twice area with single-row arrangement with same number of wells, so that capacity of wells is increased. With three-row arrangement distance between wells in longitudinal direction is $2.25D_c$, and in cross direction $1.25D_c$, so that length of collecting area is $L = (2.25n + 1)D_c$, and width $W = (2 \times 1.25 + 1)D_c$; therefore area $A = 3.5(2.25n + 1)D_c^2$. Hence for $n = 3$ and $D_c = 164$ ft., $A = 3.5\ (2.25 \times 3 + 1) \times 26{,}896 = 730{,}000$ sq. ft.

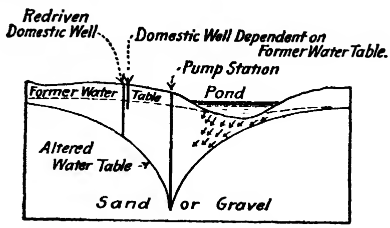


FIG. 115.—Cone of depression produced by a pump station and its effect on a near-by pond and well.

In a circular arrangement of wells, the length of collecting area perpendicular to underground stream is same as with row arrangement, namely, $0.75D_cn$, where n is number of wells on one side of center line. With respect to direction of underground stream such a ring may be considered equivalent to two rows, up-stream half of circle forming one row, down-stream the other. With respect to these rows wells are arranged at a certain distance apart. Collecting area is a ring the mean diam. of which, D_m, is the average diametral distance between centers of wells, opposite each other. Outer diam. is $D_m + D_c$, inner diam. $D_m - D_c$; hence $A = D_mD_c\pi$. Circular area enclosed within this ring is not subject to collecting effect of wells. For $D_m + D_c = 0.75D_cn$, $D_m = 0.75D_cn - D_c$; if $n = 10$ and $D_c = 164$ ft., $D_m = 1066$ ft.

It is often poor economy to place wells nearer than 50 ft. apart, and at times even 100 ft. may be the suitable distance (Mason, Water Supply, p. 339).

*König, Wasserleitungen und Wasserwerke.

Table 52. Rate of Flow in Well Pipes

Diam. of pipe, in	Flow		Number of pipes for 1 mgd
	Cu ft per min	Gals per 24 hrs	
2.5	2.0	21,500	46
3.0	3.0	32,300	31
3.5	3.8	40,900	25
4.0	5.0	53,900	19
4.5	6.2	66,800	15
5.0	7.8	84,000	12
6.0	11.2	120,600	9
7.0	15.0	161,600	6
8.0	19.5	210,000	5
10.0	30.0	323,000	3

General practice seems to sanction ordinary rates of flow in well pipes about as given above (Fanning, Hydraulic and Water-supply Engineering, 1902, p. 108*b*). Velocity is about 56 ft. per min., or 1 ft. per sec.

Table 53. Interference of Wells

THEORETICAL MUTUAL INTERFERENCE OF A GROUP OF 6-IN. WELLS, WITH RADIUS OF CIRCLE OF INFLUENCE = 600 FT., PUMPED DOWN 10 FT. (SLICHTER)

Spacing, ft	Interference, per cent		
	Two wells	Three wells	Large number in row
5	38	55	---------
10	35	51	---------
100	20	31	66
200	16	22	45
400	11	12	24
600	---------	---------	14
1000	6	8	6

Hydraulic Requirements to be satisfied by spacing are: Diameters and lengths of strainers must be such that the loss of head due to friction in them will be immaterial compared to loss in water-bearing stratum. The wider the spacing, the greater is the yield to be handled, and the greater the loss of head in pipe and strainer. Forty-two wells were sunk for Memphis Waterworks, ten being 6 in. in diam., and others 8 in. Depth varied from 260 to 480 ft., of which 80 to 130 ft. were in water-bearing strata. A 75-ft. spacing was found inadequate and 250-ft. spacing was used in later wells. (E. R., Dec. 19, 1891, p. 44; Nov. 29, 1902.)

Yield depends on both size of hole and supply. Boring is usually continued some distance below where water is tapped, in order to strengthen the supply, and the pump cylinders are placed some distance below the water level to allow for lowering by pumping.

Specific capacity is a numerical expression of the readiness with which a well furnishes water to a pump, and depends on the coarseness of the medium and the resistance offered by the strainer. The quantity can be computed by dividing the yield of the well by the amount that water is lowered in the well. Agawam wells, Long Island, discharged 1560 gals. per min. under a vacuum of 23.2 in. This corresponds to a head of 26 ft.; water in the wells was lowered but 20 ft. Specific capacity of this group of wells was therefore $1560 \div 20 = 78$

gals. per min. Area of strainer surface was 730 sq. ft. The specific capacity under 1 ft. head per sq. ft. of well strainer was therefore 78 ÷ 730 = 0.11 gal. per min. With large, properly constructed wells on Long Island and properly designed strainers, a specific capacity of 0.15 gal. per min. per sq. ft. of strainer, under 1 ft. head, should be expected. (U. S. Geol. Survey, Prof. Paper No. 44, 1904, p. 114.)

Table 54. Effects of Ground-water Pumping in Diminishing Stream Flow, Brooklyn Watershed, from 1873 to 1899, by 5-yr. Periods†
(Compiled by L. B. Ward)

Period	Average annual rainfall			Area of watershed, sq mi.	Driven well supply	
	Total, in	Collected*			Expressed as rainfall, in	Gals. daily per sq. mi.
		%	In			
1873–1877	43.3	25.1	10.8	52.3	Begun in 1883	
1878–1882	41.6	29.6	12.3	55.1		
1883–1887	43.3	31.6	13.7	64.4	2.9	140,000
1889–1893	45.1	38.4	17.3	65.5	5.8	280,000
1895–1899	43.1	36.3	15.7	66.4	7.8	370,000

Period	Other pumped supplies		Total supply from all sources on watershed (gals. daily per sq mi.)	Water collected as stream flow, referred to 50 sq mi of watershed		
	Rainfall, in.	Gals daily per sq. mi.		Gals daily per sq mi.	Expressed as rainfall	
					In	% of total
1873–1877	0.2	9,000	517,000	532,000	11.2	25.8
1878–1882	1.0	47,000	586,000	594,000	12.5	30.0
1883–1887	2.3	109,000	652,000	518,000	10.9	25.1
1889–1893	4.2	199,000	824,000	455,000	9.6	21.2
1895–1899	2.7	130,000	746,000	327,000	6.9	16.0

Testing Wells

Methods. Testing of capacity is the driller's duty and is considered an essential part of sinking, especially as tests must occasionally be made while the well is being drilled. Wells sunk by percussion methods may be tested by bailing rapidly with a bailer or sand bucket. Larger wells may be tested by centrifugal pump, and small wells by hand pump, pumping rapidly for several hours. Testing involves a knowledge and interpretation of the geologic structure. Sometimes a driller drills a well 400 or 500 ft. deep into "dry" rock; surface water drains into it, and the "test" consists in pumping accumulated surface water from a deep drill hole.

Defective Flow. Suppose two porous beds, *A* and *B* (Fig. 116), separated by an impervious layer, are to be tested, and testing of *A* has been neglected. "Suppose seed bag or rubber packing placed above the upper one. If both bear a water level equally high the test will be fairly made and the result will indicate their combined capacity; or, if both heads are at least as high as the surface at the well, the test may be accepted. But suppose bed *A*

* Referred to watershed as a whole

† For methods of measuring rate of underflow electrically, see p. 85, also E. N., Feb 20, 1902.

has been cut into by erosion or been reached by crevices or is otherwise defective, while *B* remains intact and bears an elevated fountain head. Under these conditions water may flow from *B* through the bore into *A* and escape; in this case result may be either simply negative (Fig. 116) or positively false and misleading (Fig. 117). If lateral leakage through *A* effectually disposed of flow from *B*, and there was no leakage in the upper portion of the well, water in the test tube would stand during the test at essentially the same hight as before, and result would be negative, merely failing to indicate a possibility that really existed. If there was lateral leakage through the upper strata as well as through *A*, neither alone being quite competent to dispose of the flow from *B*,

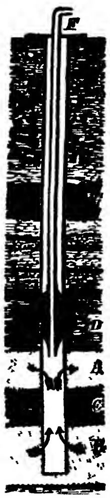


Fig. 116.—Negative test.

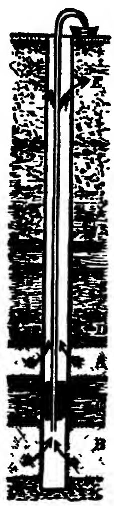


Fig. 117.—Partial and misleading test.

Fig. 118—Inverted test.

introduction of the test pipe would cut off the upper leakage, leaving *A* unable to dispose of the entire flow; there would be a rise of water in the tube and possibly a flow. A test of this sort appears to be a true test, because it shows some result, while in reality it is false and misleading. A true test in this case can be made only by placing the packing between the porous beds *A* and *B*.

"Where two porous beds, *A* and *B* (Fig. 117), have been traversed, packing placed between, then (1) if *A* equals *B* in productive capacity, water will stand at same hight within and without the test pipe if there is no leakage in the upper beds. (2) If failure to flow was due to such leakage, then a flow will result from *B*, but the additional flow which might be secured from *A* is lost. (3) If *A* has a greater head than *B*, and if there is no loss above, water in the test pipe will actually be lower than that outside, as in Fig. 118. This may be said to be an inverted test and is less misleading than the false and negative tests since it plainly indicates an error of manipulation. (4) If, however, there is in this case considerable lateral waste in the upper strata, valuable flow from *A* will be lost just as before the test was made, while *B* may give a rise in the tube, or even a flow, which would foster the impression that a fair test had been

made, while in reality the greater flow has been lost. (5) If *A* gives a feebler flow than *B*, but has an equal head, the test will fail of being completely satisfactory only in excluding the feebler flow from *A*. (6) If *A* has a lower head and is a possible means of escape for flow from *B*, then the packing has been placed at the right point and the test gives best results.

"In another case let *A* and *B* represent porous beds, lower of which is so conditioned as to drain upper by virtue of a lower outcrop. (1) If drainage loss below is not complete, and if packing is placed above *A* (Fig. 119, Well I), result will be negative if there is no leakage in upper strata. (2) Should there be considerable loss there, it will be cut off by the tube and packing, and some rise in the tube will be the result in most cases. In either instance

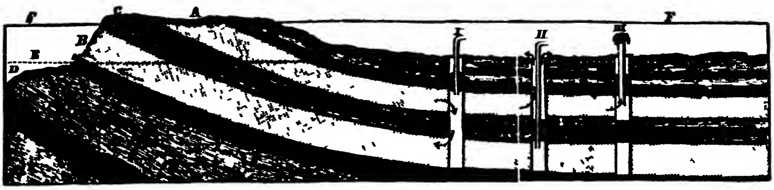


Fig. 119.—Section of strata, showing one correct and two erroneous tests.

result is misleading, particularly in the latter because the small rise of water is apt to allay any suspicion as to effectiveness of test; flow from the productive stratum is mainly lost below. (3) Suppose packing between *A* and *B* (Fig. 119, Well II): it will shut off flow from *A*, while water in *B*, because of lower outlet, will fail to flow. If there is opportunity for lateral leakage in the upper strata, water from *A* will rise in the well outside the test pipe and pass off into these open upper beds. (4) But if no such opportunity is afforded it may rise to the surface and over-flow outside the test pipe, while water within the test pipe will probably be found lower than before the test was made. Proper method of testing wells known or suspected to present these conditions is to sink a simple seed bag or other obstruction to a point in the impervious stratum between *A* and *B*, which, when it tightens, will shut off flow below. Then a tube with packing sunk at a point above *A* will effectually cut off all leakage in the upper strata, and full capacity of *A* will be tested. Test each water-bearing stratum as encountered, or else vary the final tests so as effectually to exclude all liabilities to error."*

Measurement of Depth of Wells

Cable Measurements. With a standard outfit depth is usually measured by the cable as the tools are lowered. Just as top of tools is about to enter the hole, a string is tied to the cable at the bull-wheel shaft; the tools are lowered until this string has gone up over the crown pulley and down to the well head, then another string is tied to the cable at the bull-wheel shaft, and so on until the tools reach the bottom. Number of strings tied to cable multiplied by distance from bull-wheel shaft up over crown pulley and down to well head, plus last fraction of this distance gives total depth of well.

* T. C. Chamberlain: "Requisite and Qualifying Conditions of Artesian Wells," Fifth Annual Report, U. S. Geol. Survey, 1885.

Errors may be caused by slipping of strings, by stretching of cable, or by miscount.

Tape Measurement. Measurement with a steel tape has often been found difficult, on account of magnetized condition of casing, produced by jarring of the drilling tools. To avoid difficulties of magnetization a copper wire or tape may be used.

A double-cone devised by J. E. Bacon for measuring depths and thicknesses of water-bearing strata contains a central brass plate, over whose edge are drawn wires fastened at each end to a center post. To one end a line is attached. As it is lowered into the hole a steady stream of water under known pressure is forced down the well. This acts on the brass plate and produces a determinable tension on the suspending line. When a water-bearing stratum is reached loss of water into it and consequent decrease of tension on the line as the instrument passes its upper surface at once indicates its position. A reverse change of slighter value as the instrument passes the bottom of the stratum in like manner indicates this second position.

OPERATION OF WELLS

Restoring Lost Supplies. When a deep well is first sunk, it usually gives good supplies, but as time elapses the yield decreases until it is but a small fraction of the original; this is commonly attributed to a decrease in the general supply of the region or the drawing off of the water by newer and better wells. In many wells this is the real cause, but in others failure is due to deterioration of the old well. The chief causes are clogging of the screen, entrance of sand, and leakage, of which the first is probably the most common. Clogging may result from filling perforations by sand and silt, or by deposition of iron or lime. Where waters carry acids, the screen is often corroded, the iron set free being redeposited in the sand about the screen, forming impervious coatings. When matter collected about a well is soft and loose, it can often be removed by pumping heavily into the well. When this fails, the usual remedy is to pull the casing and screen, or the pump and well point, and replace.

Interference by Salt Water. Some supplies pumped from wells near the seashore have been seriously affected by salt water, although the normal course of underground water is from the land toward the sea. At Spring Creek station, Brooklyn water supply, the ground waters had become so exhausted by 1897 that there were 300 parts per million of chlorine against a normal of 5. Plant was shut down. One well contained 1400 parts. Jamaica Bay, about 1¼ mi. from the wells, contains 10,000 to 15,000 parts. At New Utrecht plant, chlorine rose from 20 to 100 parts, following a rise in pumping rate. Chlorine has increased with time at Shetucket Wells, L. I.,* as follows:

Date	Chlorine, parts per million	Pumping rate, mgd.
Oct, 1896 to Mar., 1898 .	5	3 7
End of 1898 ..	74	6 for 1 month; then 3.5
Oct. 1899 .	246	2.0
End of 1902	500	1 25

*In 1896 twelve 8-in. wells were put down 175 ft. deep.

At end of 1902, pumping rate was reduced to 0.5 mgd., but chlorine remained high and the station was abandoned.

Experience shows that several months and sometimes years elapse between beginning of heavy pumping and appearance of chlorine. It is possible that salt water, being heavier, forces out the fresh water for the full depth of the saturated stratum, and sea water gradually advances toward the wells as the fresh water is withdrawn. Capillary action will hold much fresh water in interstices of the gravel, which will dilute the salt water enough to cause the difference between the sea and spring water shown in the analyses; this accounts for the gradual increase in chlorine. After pumping has exhausted fresh water, it would be necessary to close stations several months, if not years, to give the fresh water opportunity to drive out the salt water between the station and tide water. It is therefore impractical to continue operating a station after chlorine has passed the danger mark.

Packing with Gravel or Coarse Sand. Frequently, where the material is very fine, it packs around the well so as to hinder entrance of water. It is common practice to drop pebbles into the well and with the aid of a drill, force them into the surrounding clay, etc., until a pocket is produced through which water flows freely. In quicksands, the space between the outer casing and the pump tube and strainer is often filled with coarse sand through the heavy suction induced by pumping. Many materials yielding water consist of mixtures of sand or gravel and clay. By heavily pumping a new well it is often possible to remove fine clayey material, leaving sand grains and pebbles in a pocket about the well. A similar method is employed in certain stiff clays in which small open pockets at the bottom of the well are apparently produced by heavy pumping.

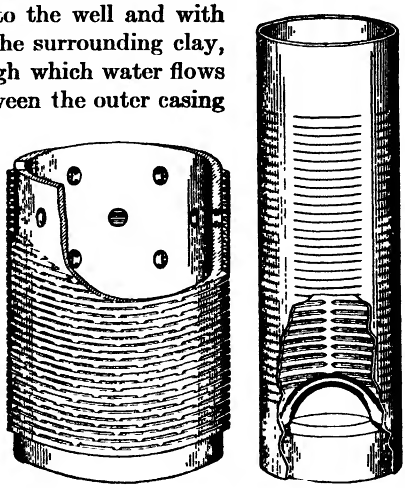

1 2

FIG. 120.—Well strainers.
1. Layne strainer 2 Cook strainer

Well Screens or Strainers. A screen in use in the Mississippi valley from St. Louis to the Gulf is made by wrapping No. 14 wire, 10 to 14 wires to the inch, around perforated wood or iron piping; the closeness of the wrappings depends on fineness of the material to be screened out. Wooden piping as a base for a screen is often preferred on account of rusting of iron pipe. In some wells these screens are 100 ft. long. The weight of the wire screen and the sand pressure against it when set prevent the buoyancy of the wood from lifting the pipe. Several patented strainers* are also used. The Layne† strainer differs from shop-made ones in the shape of wire. The Cook‡ well screen consists of a single piece of seamless brass tubing, perforated by horizontal slits, in different widths for different sizes of material, cut with a beveled edge on the inside. The screen may be set in the bottom of the well, the pipe drawn until it is almost flush with the top

* "Johnson" strainer is made by American Well Works Co., Aurora, Ill
† Layne & Bowler Co., Houston, Tex.
‡ A. D. Cook, Lawrenceburg, Ind.

of the screen, and a lead seal inserted at the joint, or the screen and pipe may be united by a screw coupling.

Straimers may have either: (1) Fine openings to exclude the finest sand; or (2) openings somewhat larger than the finer particles in the water-bearing material, and smaller than the coarser. Pumping soon exhausts the finer material next to the strainer, and the coarser particles form what is really a greatly enlarged strainer outside the metal one. If no coarse particles exist, ram some down outside of casing. Such a well cannot clog. Any strainer depending on wire gauze or thin perforated metal to exclude fine sand is undesirable, for friction is high in small openings in sand and screen; and corrosion will close up the small holes in the metal. (Maury, Proc. Am. W. W. Assn., 1904.)

Corrosion of Tubular Wells. In Victoria, South Australia, the iron lining of a bore was eaten away so as to destroy its continuity in 18 months. There was a saline constituent in the water. (J. C. Thrush, "Water and Water Supplies," 1901.)

Life of a casing cannot be definitely predicted, rate of decay depending on conditions in each well. Casing withdrawn from some wells 15 to 20 yrs. old has been found in fairly good condition except at joints, though as a rule at this age it is too badly corroded to be withdrawn at all.

Detection of leaks is somewhat difficult. In some wells water may be heard trickling in or seen by a light ray projected by a mirror when the pump is withdrawn. The admixture of water from outside may sometimes be detected by a difference of hardness, by taste, or by cloudiness due to silt. The remedy is usually to pull the old casing and replace it by new. The time the pipe is allowed to remain before replacement is determined by estimate based on the action of water on the pump tubes or other pipes. An alternative treatment, when the leak is near the surface, is to set a packer in the space between the bottom of the pump tube and casing and fill space above with cement.

WELL CURBS, CASINGS, AND COVERS*

Dry rubble curb and casing utilizes all seeps. The material costs little; little money outlay for labor. The well is never safe near sources of contamination. Affords no filtration and allows dirt and soil to enter. Permits entrance of mice and other small animals at the top. Do not use stones partially or wholly covered with moss or lichen; persistent impairment of the quality of the water has resulted from such use.

Dry brick curb and casing utilizes all seeps. Filters out most sediment. Does not allow small animals to enter. Involves little money outlay for labor. Polluting matter enters readily, and the well is never safe near sources of contamination.

Curb and Casing of Stone or Brick in Cement. The well is safe from pollution (except that entering at bottom) as long as walls are not cracked. Entrance of sediment and animals is prevented. Wall does not impart taste to water. Utilizes water from bottom only. Well is unsafe if so shallow that polluting matter can reach its bottom. Costs more than uncemented

* See also page 654

wells; may require skilled labor. A common practice is to use cement in upper part but not in lower, where water of satisfactory quality is admitted.

Wooden curb and casing, is cheap in many localities. Can be used in wells of small diam. It imparts, when sound, no taste to the water. It swells tight in wet ground, water either entering at bottom, or (after sudden rises) through shrunken portion at the top. Pollution enters readily, and animals gnaw through. The wood rots, giving taste to water and favoring development of bacteria.

Glazed and Cement Tiles. (1) With uncemented joints. Utilizes all seeps. Imparts no taste to water. Requires no skilled labor. Polluting matter enters readily and well is never safe if near source of contamination. Soil may wash in through joints. (2) With cemented joints. Well is safe from pollution (except that entering at bottom) as long as joints are tight. Does not require expensive labor. Can be used only in soft materials containing considerable water.

Iron casings are adapted both to rock and to unconsolidated materials. Safe from pollution except that entering at bottom. The cost in large deep wells is considerable. Use is practically limited to wells under 14 in. diam. In some places, it is subject to deterioration by corrosion and incrustation. Utilizes only bottom water stratum, except where perforated. See also page 222 for California wells.

Combination dug and drilled well is particularly dangerous, because of the fancied security. A drilled well is sunk in an old dug well, the casing commonly beginning at the bottom of the old well. The casing should be carried to surface of the outside ground, or at least above highest water level, or the dug well should be converted into a water-tight cistern.

Covers. Open wells should be protected by a water-tight iron or cement cover standing somewhat above the surrounding ground, and tightly joined to the curb; sloping the earth away from the well serves to run rain-water or pump drippings away, so that little will penetrate, even if the curb becomes cracked by frost. Except that it keeps the larger animals out, ordinary plank covering affords but little improvement over the open well. Crevices almost invariably exist through which small animals find access, and dirt washed through the cracks is of the most dangerous kind, often filth from domestic fowls, and from the shoes of farm hands. (See Fig. 346, p. 656.)

WELL PUMPS

(See also Hot-air Engines, p. 497.)

For pumping from deep wells, some form of power pump is more economical than the direct-acting steam pumps. Power pumps may be driven by electric motors, gasoline, oil or steam engines or water power. Gasoline pumping plants have been extensively used since 1900 and are giving satisfaction for small plants, but where considerable water is to be handled, it can be done more cheaply by steam. For small installations hot-air engines are quite satisfactory.

Deep well centrifugal turbines are made for pumping water from wells whose diams. to standing water are 12 in. and larger; they possess the advantage of having all machinery except the turbines above ground. Where

water is obtained from deep wells for fire protection, the best practice is to install a centrifugal turbine to raise water to the surface and deliver it to a single or multiple-stage centrifugal, to force it to the elevation required. Where water is charged with more or less sand or rock cuttings, the turbine is best adapted for pumping it, as the bearings are protected by an inner tube, and there are no packings or leather cups to be cut out as in the case of reciprocating cylinders.

Lack of space prevents discussion of Well Pumps and Fittings. See catalogs of American Well Works Co. (Aurora, Ill.); Goulds Mfg. Co. (Seneca Falls, N. Y.); Geo. E. Dow Pumping Engine Co, (San Francisco).

AIR-LIFT PUMP

Definition. The air lift, or air-lift pump, is an apparatus for raising water from wells by means of compressed air forced to relatively great depth in the well, through a pipe, and so discharged as to mix with the water in small or large bubbles.

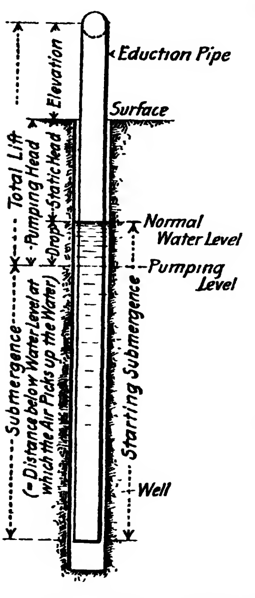


FIG. 121.—Nomenclature of air lift pump. (Sullivan Machinery Co)

Principle of Air Lift. As compressed air enters discharge pipe near bottom at pressure only slightly above hydrostatic head, column of water above is forced upward; mixture of air with water lessens its specific gravity and aids upward motion. Air continues to enter, taking place of rising body of water, until water flows from discharge opening. The moment that part of the rising water is discharged, weight of column becomes less, and air beneath will correspondingly expand, thus reducing pressure on water in discharge pipe below air inlet. Weight of water in well outside of discharge pipe then forces water into discharge pipe, stopping inflow of air. Pressure in air-supply pipe is quickly renewed by its connection with supply, so that it again forces entrance into discharge pipe. This process is repeated until whole discharge pipe, above air inlet, is filled with alternate bodies of air and water, the combined weight of which is enough less than water in well to keep up constant flow of water into discharge pipe. Air issues at mouth at atmospheric pressure, expanding as each succeeding layer of water above it is discharged. Principal loss seems to be slipping back of water layers due to friction of pipe.

Symbols.*

a_p = area of eduction pipe in sq. ft.
c_e = coefficient of entrance.
c_p = coefficient of pipe friction and average slip.
h_l = lift, ft.
h_s = depth of submergence, ft.

* The following pages are abstracted from Bull. No. 450, Oct., 1911, Univ. of Wisconsin, Davis & Weidner, unless otherwise credited.

l = work output, ft.-gals. per sec.
p_b = barometric pressure, acting on the surface of the water in well and also on the discharge end of pipe.
d = diam. of eduction pipe, ft.
p_i = absolute pressure at inlet in foot-piece.
q_b = discharge of air at pressure p_b.
q_w = discharge of water, cu. ft. per sec.
u_w = density of fluid pumped.
v_i = velocity of the liquid in the eduction pipe below the air inlet.

Lorenz's Theory. A simple mathematical theory of the air-lift pump was published in Zeitschrift des Vereines Deutscher Ingenieure, Vol. 53, p. 545, Apr., 1909. The formulas take account of losses of energy occasioned by slip, pipe friction, etc., and are of practical use in designing pumps, provided the necessary experimental coefficients are known.

During operation of pump, the following equations of heads hold between point c (Fig. 128(a), p. 252) in pump and a point at same elevation outside:

$$h_s - \frac{p_i - p_b}{u_w} = \frac{v_i^2}{2g}(1 + c_e) \tag{1}$$

$$\frac{q_b p_b}{q_w u_w} \log_e \frac{p_i}{p_b} = h_l + \left[\frac{\left(1 + c_p \frac{l}{d}\right)(q_b + q_w)^2 + c_e q_w^2}{2ga_p^2}\right] \tag{2}$$

$$\frac{1}{q_w}\frac{p_b}{u_w} \log_e \frac{p_i}{p_b} = \frac{1 + c_p \frac{l}{d}}{2ga_p^2}(q_b + q_w) \tag{3}$$

$$\left(1 + c_p \frac{l}{d}\right)(q_l^2 + q_w^2) = 2gh_l a_p^2 + c_e q_w^2 \tag{4}$$

If the maximum discharge, determined from the capacity of the well, and the area a_p of discharge pipe, determined from the diam. of the well, and also the lift and the known coefficients c_e and c_p are given, the volume of free air required may be computed by formulas (3) and (4), from which the submergence h_s can then be computed by equation (1). Equation (2) then gives the relations between any desired values of q_b and q_w using the same pressure p_i.

Method of Operation. To start requires a greater air pressure than for normal operating. When air is first turned on, pressure must be greater than that due to the submergence of the air inlet; after discharge of liquid has commenced, pressure at the air inlet will be reduced by the amount of entrance and velocity heads of liquid entering the eduction pipe. When a sufficient number of bubbles have been introduced to raise the head through the entire lift, some liquid will begin to spill over top of pipe. The loss of this liquid causes a reduction in pressure in the eduction pipe, which under some conditions allows a sudden influx of the high-pressure air, resulting in a violent discharge of the liquid and air which may exhaust the store of compressed air. Following this, the liquid would regain its full static head, requiring a new start. To prevent such action, the escape of air into eduction pipe should be throttled the instant discharge of liquid commences. That intermittent action does not always occur is due probably to effect of friction in the air pipe. Since

friction increases as the square of velocity in long pipes of small cross-section it will serve to some extent as a governor.

Eduction Pipe. The discharge pipe (eduction pipe, lift tube, lift pipe, or rising main) should not touch the bottom of the well or reservoir, but should freely admit water through its lower open end. This end should be submerged a distance greater than the hight above water surface to which the liquid is to be lifted, called lift of the pump. The distance from the water surface down to the point of admission of air into the eduction pipe is called submergence of pump, and is generally expressed as a percentage of the total length measured from air inlet to point of discharge. Discharge should be free into a reservoir at atmospheric pressure. Submergence and lift should be measured under working rather than static conditions. Figs. 122 and 123 show various methods of piping wells.

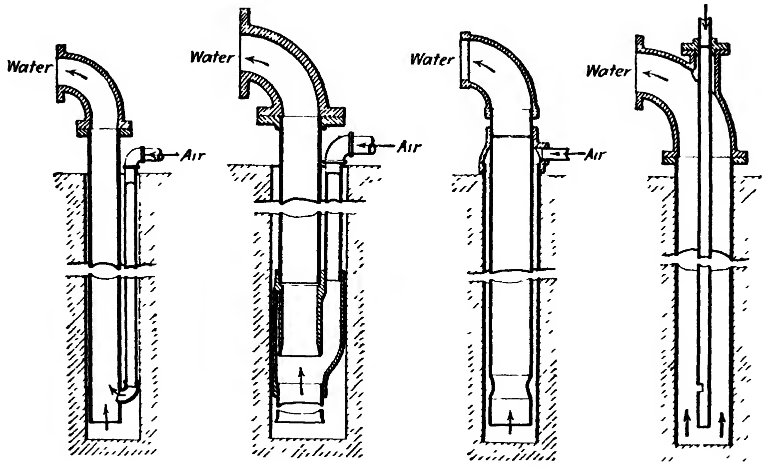


FIG. 122.

Table 55. Ratio of Lift to Submergence for Air-lift Pump

Lift	Per cent. submergence	Lift	Per cent. submergence
Up to 50 feet.......	70 to 66	200 to 300 feet.......	50 to 43
50 to 100 feet	66 to 55	300 to 400 feet.......	43 to 40
100 to 200 feet	55 to 50	400 to 500 feet.......	40 to 33

Sullivan Machinery Co.

Multiple Air-lift Pump. When the lift is very high, and the proper submergence difficult to obtain, the arrangement in Fig. 124 may be used where cross-section of well permits. This employs a series of successive lifts; it is claimed to work more economically than a single lift.

Return Air Pump. In rising through the eduction pipe there is a transfer of heat between the air and water; the temperature of the two being prac-

tically equal at the point of discharge. Therefore when pumping from underground supplies, air from discharge pipe will be cooler than the atmosphere during the warm months of the year. For each 5° fall in temperature

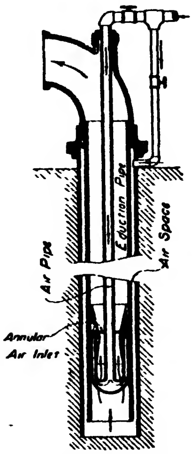
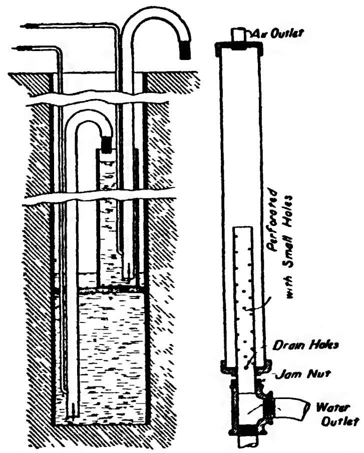


Fig. 123.—Combination air lift.



Multiple air Lift. Air separator.
Fig. 124.

of the free air entering the compressor, a saving of 1 per cent. in energy in compression may be effected. Hence, where wells are close to the power house, economy may be effected by connecting the inlet of the compressor with the top of the well casing head. A separator for this purpose consists of a cylindrical drum about 18 in. in diam., 8 or 10 ft. long, attached to the casing head, as in Fig. 124.

Diverging Outlet Pump. When the eduction pipe is of uniform diam. throughout, discharge occurs at high velocity, resulting in a considerable loss of energy. To conserve this kinetic energy, Jos. Price used an eduction pipe which increased in diam. toward the top so that as the air expanded the velocity would not be greatly increased. This device can be used with any foot-piece and any method of piping. Experiments indicate saving.

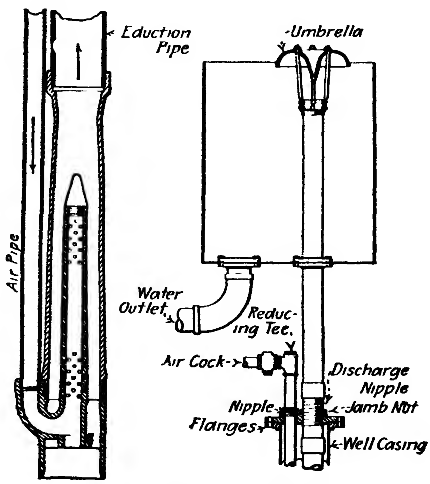


Fig. 125.—Sullivan standard air lift pump and umbrella well head, with casing flanges and connections

Disadvantages of the Air Lift Pump. (1) Low efficiency. Generally credited with only 25 to 33 per cent., but notwithstanding low efficiency of the pump itself, the entire plant in some cases developes a duty which compares

favorably with other systems of pumping. (2) Great depth of submergence. A single air lift pump cannot be used in a shallow well or reservoir, except to raise liquid a small distance, owing to the high percentage of the total length of the pump which must be submerged to give good efficiencies. This limits the air lift principally to deep well pumping. The multiple stage pump overcomes the difficulty, but probably at reduced efficiency. (3) Limited horizontal pumping. Several plants have been installed to pump a considerable horizontal distance, but such plants are not considered efficient. The air

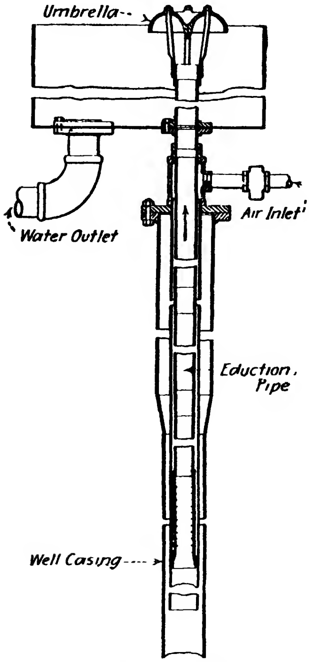


FIG. 126.—Sullivan special air lift pump and umbrella well head.

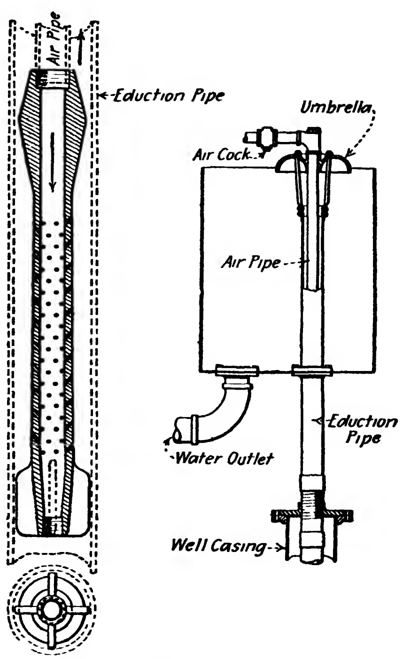


FIG. 127.—Sullivan central air lift pump and umbrella well head, with casing flanges and connections.

in passing through a horizontal or even an inclined pipe, is not likely to be evenly distributed throughout the cross-section, but to pass along the upper side, allowing a large space in the lower portion for the water to slip back past the bubble. In a horizontal pipe the air cannot exert any buoyant effort to aid in discharging the water, and its expansive force, which might be used in overcoming pipe friction, is not likely to be effective on account of the serious slip. Where it is desired to convey water to a point distant horizontally from the well, the eduction pipe should be carried vertically to a hight equal to the friction head in the horizontal conductor, and at its top fitted with an air separator. (4) Aeration. The thorough aeration of the water pumped is generally regarded as an advantage, but under some circumstances it promotes rusting and consequent destruction of the eduction pipe and in some cases causes a deposit of salts which clogs the passages, especially in the foot-piece. The opinion has also been expressed that compressed air

causes an excessive growth of algæ; the bacterial content of water is somewhat increased by the air lift unless the air supply is filtered.

Advantages of the Air Lift Pump. The air lift pump possesses decided advantages when applied to certain conditions: (1) Large capacity. When conditions are suitable, an air lift pump will discharge more liquid from a well of small bore than any other type of pump, due to the fact that almost the entire cross-sectional area of the well is available for the flow of liquid, and the action is nearly continuous. The air lift affords a means for testing the capacity of a well even if it is not to be permanently installed. (2) Low maintenance cost. Owing to the simplicity, the cost of maintenance is very low; the life of the pump is almost indefinite. Sometimes air pipes and the foot-piece become clogged with oil carried over from the compressor cylinders and have to be removed and cleaned; this rarely occurs, and the cost is small compared with the cost of replacing a mechanical pump. The absence of moving parts in the well makes the pump especially fitted for dirty water, sewage, mine water, acid or alkaline solutions in chemical or metallurgical works, or other corrosive liquids. Liquids that attack metals, such as brine, sulphuric acid, etc., may be pumped by the air lift, because the pump and appurtenances may be replaced at small expense and loss of time. The air lift as a dredge pump has been successfully but not extensively used. (3) Low operating cost. Where wells are scattered, or remote from the power house, the air lift has an advantage over the steam-driven pump. In a deep-well pump driven by steam each well must be equipped with a separate engine and working barrel, which entails heavy condensation losses through long steam supply pipes; expense of attendance is great. In the air lift pump, the transmission loss is much smaller; no attendance at the well is required, operation being controlled by a valve in the power house. (4) Not affected by high temperatures. Fluids of different densities and temperatures may be handled to advantage where other types of pumps would be prohibited. In a hot liquid, air absorbs part of the heat and is increased in volume, so that the discharge for the same expenditure of free air is greater with hot than cold liquids. This results in a considerable gain in efficiency for the pump. (5) Aeration. Iron is oxidized by aeration and the supply is thereby improved. Aeration is especially advantageous in pumping sewage. (6) Reliability. Air pumps are not liable to sudden stoppages or breakdowns.

Use of air lift* should be limited to cases where efficiency can be sacrificed for sake of reduction in maintenance expense, increase in well output or increase in reliability, to crooked wells or those in which water must be pumped from a greater depth than 200 ft.

Efficiency* of even a well-designed air lift is low; varying from 20 per cent. for a lift of 600 ft., to 45 per cent. for a lift of 50 ft.; and is greatly influenced by ratio of submergence to lift; the best ratio being about 2. Maximum efficiency was secured when submergence was 2.5 times lift, and efficiency fell off more rapidly for a reduction in submergence than for increase. Test was not carried to low enough submergence to reach maximum duty; duty of this lift would have been increased 42 per cent., and cost of pumping reduced that amount, had well been operated at a flow of 700 gal. per min. instead of 1100.

*E. C., Mar. 17, 1915.

Table 56.* Effect of Submergence†

Submergence, ft.	Efficiency, per cent.
8 70	26 5
5 46	31 0
3.86	35 0
2.91	36 6
2 25	37 7
1 86	36 8
1 45	34 5
1 19	31 0
0 96	26 5

Flow about 1,100 gals. per min.
Lift about 37 ft.

Table 57.* Air-lift Tests to Determine Best Flow from Deep Wells, Hattiesburg, Miss.†

Length of air pipe, feet	Flow gal. per min.	Lift feet	Efficiency, per cent.	Duty gals. per hp.-hr
224	1,495	47.5	32 6	1,630
208	1,459	45 8	31 5	1,640
184	1,419	44.3	31 1	1,670
162	1,359	39.4	30 0	1,790
142	1,299	37.7	30 6	1,920
124	1,219	37.5	32 8	2,070
105	1,106	37 1	36.7	2,340
86	1,008	32 5	33.4	2,450
79	904	29 3	31.3	2,530
67	802	26 0	30.5	2,750
43	690	21 2	29.8	3,240

Reasons for Low Efficiency.‡ So little difference in pressure exists between air entering eduction pipe and water that the air flows into the water at low velocity and must travel some distance toward surface before it has expanded sufficiently to form a plug and carry water with it, thus sacrificing part of submergence. 2. As bubble travels toward surface and pressure above decreases, it must expand, and being confined in the pipe, can do this only in a vertical direction—resulting in increased velocity and friction and greater displacement in the eduction pipe. 3. Flow at contact between a gas or liquid and walls of passage is retarded, and water, being heavier than air, is retarded more by friction until it is pulled back around the bubble. Thus bubble becomes elongated and at times slips through, joining the preceding bubble and leaving the water behind. This slippage represents loss of efficiency and is manifested by variations in the plugging discharge. 4. Air, like any confined gas, is always seeking a chance to escape, and any inequalities in the flow passage or abrupt changes in direction, offer the opportunity sought, resulting in further slippage.

Requirements for Efficient Air Lift.‡ 1. Means to secure a perfect mixture of air and water into an emulsion at point at which air is injected into water. Then each particular small bubble will start its lifting effect at once. 2. A Ventura, or choke, just above the mixer. This will increase the velocity and give a jet effect at this point. 3. Eduction pipe arranged with proper enlargement, to allow for expansion of air so far as possible and to prevent excessive velocity toward discharge. 4. An absolutely smooth passage for the air and water. Even the swirl caused by the recess in a coupling, occurring as often as it does in a long pipe, will cause much loss. 5. Proper proportioning of air and water pipes.

Tests. While there have been many tests on actual wells, facilities for varying the conditions of operation or for accurate measurements have been limited; comparative tests therefore have been confined to laboratories.

* E. C., March, 17, 1915.
† "Pumping by Compressed Air," by E M Ivens. Speed of air compressor was adjusted so as to keep rate of flow of water constant, while length of air pipe was varied; dimensions of well and air lift were as follows:

Total depth of well, ft.	453 5
Inside diameter of casing, in	9½
Inside diameter of air pipe, in	2½
Inside diameter eduction pipe, in.	9½
Static lift, ft	3 to 4.0

‡ Sullivan Machinery Co.

From experiments on actual wells, Prof. Josse deduced the following: (1) If submergence and lift be constant, the amount of air per volume of water will not vary much with size of pump. (2) With increasing lift, the volume of air per volume of water increases, and hence efficiency decreases. (3) Other things being equal, an increase in the area of discharge pipe of 20 per cent. increased the discharge only 1.2 per cent. Efficiencies varied from 20 to 45 per cent. in the laboratory tests; from 22 to 28 per cent. in operating wells. Kelly experimented at Preesall, Lancashire, on actual wells. Indicator cards were taken from the steam and water cylinders. Efficiencies were calculated from the ratio of the work done in raising water to the work indicated in the air cylinders of the compressor. The same general result was obtained in efficiency, discharge and ratio of volume of air to volume of water, whether one well was working, or two or three together. One well was piped according to the side inlet system, and three wells according to the annular tube system, air entering the eduction pipes through their open ends. The size of air and eduction pipes varied. In the well piped according to the side inlet system, the upper end of the eduction pipe was enlarged from 4 to 6 in. in diam., to reduce the velocity of discharge. The total length of the eduction pipe varied from 323 to 433 ft. The highest efficiency was about 40 per cent.,* obtained when three wells were working together. Conclusions: (1) Highest efficiency was obtained at lowest rate of working. (2) Discharge increases as the rate of working increases, with a tendency to decrease after the rate reaches a certain point. (3) A submergence of 60 per cent. gives a better efficiency than of 50, other conditions remaining the same. (4) A well piped with 5-in. eduction and 7-in. air pipe gave greater efficiencies than one piped with a 4-in. eduction and a 6-in. air pipe, other conditions remaining the same. (5) A well pipe with the side inlet system and a diverging outlet appeared to give better results than other wells, the cross-sectional area of the eduction pipes remaining equal. (6) The action of the compressed air in a lift may be similar to that of a piston in a cylinder, may form an emulsion with water, or may produce a combination of both, the result depending on the rate of working. Piston-like layers are obtained with the higher rates of working.

Tests of Westinghouse Air Brake Co. Nearly 1800 experiments, covering nearly 400 different combinations of discharge pipe, diam., lift and submergence, were made on an actual well, 6 in. diam. and 174 ft. deep. Following conclusions are from Eng. News, June 18, 1908: (1) The rate of delivery of water, and the air consumption per gallon, with fixed size of discharge pipe, are practically constant for all lifts, provided the ratio of lift to submergence is maintained constant. (2) With a discharge pipe of given diam., the delivery decreases and the air consumption per gallon increases as the ratio of lift to submergence increases. (3) With a fixed ratio of lift to submergence, the air consumption per gallon decreases as the size of discharge pipe increases. (4) The least air pressure that will give continuous flow is the proper pressure to use. A slightly lower pressure gives intermittent delivery, and the amount is much decreased, though the air consumption per gallon is slightly lower than with continuous flow. With pressure higher

* This is the highest efficiency by actual test known to the Handbook editor.

than required to give continuous flow, delivery is increased somewhat, but the air consumption per gallon delivered is increased in greater ratio; and with further increase in air pressure, a point of maximum delivery is reached beyond which the delivery is decreased. The sound of the discharge is a reliable guide to the proper regulation of air supply. (5) It appears from (2) that by increasing the submergence, *i.e.*, locating the foot-piece deeper in water, for a given lift, the air consumption is progressively reduced; but as the required air pressure is increased, a cubic foot of air represents greater power. A curve representing the variation of horsepower required per gallon of water delivered, with depth varying, shows that the power first decreases with increasing depth, then reaches a minimum, and thence increases. The ratio of lift to submergence at this minimum point may be called the "economical ratio." (6) For a given size discharge pipe, the economical ratio decreases as the lift increases; *i.e.*, the submergence should be increased in greater ratio than the lift. For a given lift, the economical ratio increases (submergence decreases) as the size of discharge pipe increases. (7) A tail-piece or projection of the discharge pipe below the air inlet is essential in starting, as it tends to prevent air from backing down into the well and rising in the casing outside the discharge pipe. (8) Any jet or pipe introduced into the discharge pipe to serve as an air inlet has no value, and is detrimental by forming an obstacle to the free passage of water. (9) The size of air pipe is determined only by considerations of friction loss required to force air through the pipe.

Duty Tests. The published results of duty tests are few. The plant is usually so arranged that a duty test is difficult, as the air compressor takes steam from the same main as the force pumps. The plant at Atlantic City,* N. J., showed a duty ranging roughly from 20,000,000 to 25,000,000 ft.-lb. per 1000 lb. of dry steam; the plant consisted of a Rand duplex flywheel compressor, having 10-in. and 16-in. cross compound steam cylinders of 12-in. stroke, 13 by 12-in. air cylinder, Corliss inlet valves, and poppet discharge valves; a compressor of the same type with 11-in. and 18 by 14-in. stroke steam end, and 16 by 14-in. air end, having Meyer adjustable valves on the steam cylinders; a Wainwright surface condenser; a Deane combined wet vacuum and circulating pump, with $5\frac{1}{2} \times 7$ in. steam end and 6×7 in. pumps; an air receiver 28 in. by 8 ft.; and galvanized piping to the wells, with valves. The smaller compressor was to be in reserve. The contract price for the pumping equipment was $8250. There were thirteen wells with a combined capacity of 5,450,000 gals. per 24 hrs. The average lift is about 27 ft., and submergence about 60 per cent.

Wells pumped with air lift and steam deep-well pumps at Waukesha, Wis., showed an efficiency between 16 and 18 per cent. for air lift based on indicated hp. in the steam cylinder, and 74.8 per cent. for the deep well pumps based on indicated hp. of the engine.

	Air lift	Deep well pumps
Duty per 100 lb. of coal	8,200,000	34,500,000
Duty per 1000 lb. of steam	11,940,000	53,300,000

University of Wisconsin Experiments. Variables which may affect a particular size and type of air-lift pump are: (1) percentage of submergence; (2) lift; (3) discharge; (4) volume of air; (5) pressure of air. Conclusions from

* Tested in 1904 See E N., June 18, 1908.

the Wisconsin experiments are given below, and hold only for the type, size and length of pump on which the experiments were performed. The inference may be drawn that these conclusions would hold for other types and sizes. (1) The central air tube pump has the greatest theoretical capacity for a given size of well. (2) The coefficient of pipe friction and slip decrease as the discharge increases, and decrease as the ratio of volume of air to volume of water increases. (3) The coefficient of pipe friction and slip varies with the length of pump, but seem to be independent of the percentage of submergence and of the lift. (4) The length of pump, the percentage of submergence, and therefore, the lift remaining constant, there is a definite quantity of air causing the maximum discharge. This quantity, as also the ratio of volume of air to volume of water, differs for different percentages of submergence and lift, the length of pump remaining constant. (5) The length of pump remaining constant, the maximum output (*e.g.*, ft.-gals.) occurs at about the same percentage of submergence for all rates of air consumption, being from 61 to 65 per cent. for Wisconsin experiments; at other submergences the output varies as the ordinates of a parabola having a vertical axis. Under these conditions the lift does not remain constant as the percentage of submergence varies. (6) The length of pump and percentage of submergence remaining constant, and therefore constant lift, the efficiency increases as the input decreases, that is, the highest efficiencies are obtained at the lowest rates of pumping. (7) By varying the percentage of submergence, and therefore the lift, the length of pump remaining constant, the maximum efficiency is obtained at approximately 63 per cent. submergence for all rates of input and discharge. (8) The lift remaining constant, the efficiency increases as the percentage of submergence increases, for all rates of input and all practical percentages of submergence. (9) With the same size and type of pump, the percentage of submergence remaining constant, the efficiency increased as the lift decreased for the small lifts experimented on (up to about 24 ft.) From a theoretical study, the indications are that a point will be reached beyond which the efficiency will decrease as the lift increases. (10) Other conditions remaining constant, no advantage is to be gained by introducing compressed air above the surface of water in the well. (11) The type of foot-piece has little effect on the efficiency of the pump, so long as the air is introduced in an efficient manner, and the full cross-sectional area of the eduction pipe is realized for the passage of liquid. Anything in the shape of a nozzle to increase the kinetic energy of the air is detrimental. (12) A diverging outlet which will conserve the kinetic energy of the velocity head increases the efficiency.

***Frizell System** attributes action of air lift to aeration of water in discharge pipe (intimate commingling of air with water), which may be sufficient for moderate lifts, that is, for cases in which the water rises nearly to the top of the well.

Pohlé System, applicable to lifts greater than 25 ft., makes use of alternate piston-like layers of water and of air formed in the discharge, or eduction, pipe. With intelligently designed air and water pipes and a plant skilfully adapted to the given wells, economy is claimed as compared with many other systems of

* Following paragraphs are largely from "Water Lifted by Compressed Air," Ingersoll-Rand Co., 1912.

pumping. Plants with lifts up to 1200 ft. are said to be in successful operation. Special plants are working satisfactorily with submergence less than 30 per cent.; and some with over 75 per cent. Percentage of submergence is the percentage of the total length of the air pipe submerged in "solid" water while the pump is operating; for a lift of 20 ft. it should be 66 per cent., for a lift of 500 ft., 41 per cent.; for average conditions, between 50 and 65. Submergence governs starting and working air pressures required. Quantity of free air, cu. ft., to be compressed, to lift 1 gal. of water = lift in feet ÷ C × log [(submergence in feet + 34) ÷ 34]. For

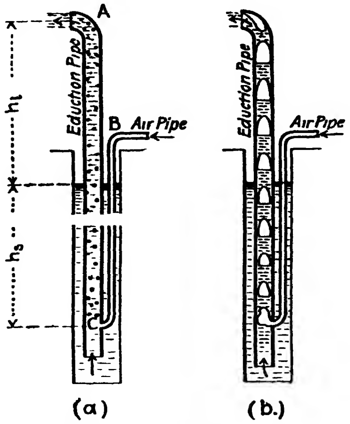


FIG. 128.—Comparison of Frizell and Pohlé systems of operation; h_s is greater than h_l.

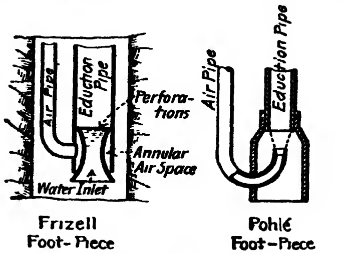


FIG. 129.

Lift =	10–60,	61–200,	201–500,	501–650,	651–750 feet
C =	245	233	216	185	156

Table 58. Pohlé Side-inlet Air Lift: Pipe Sizes and Capacities

Air pipe connection, in	Water pipe, in	Size well, in	Maximum economical capacity on moderate lift, gal per min
½	1	3	7
¾	1½	4	20
1	2	4½	35
1	2½	5	60
1¼	3	6	90
1½	3½	7	120
1½	4	8	160
1½	5	9	250
2	6	10	350

Table 59. Pohlé Annular Foot-piece Air Lift: Pipe Sizes and Capacities

Well, in	Pipe sizes			Max economical capacity, moderate lift, gal. per min	Dimensions of foot-piece	
	Disch., in	Air, in	Tail, in		Outside diam., in.	Length, in
4	1½	½	2	20	3 11/16	11
5	2	¾	2½	35	4 19/32	12
6	2½	1	3	60	5¼	12½
6	3	1	3½	100	5⅞	13
6	3½	1¼	4	140	5 13/16	13
8	4	1½	4½	190	7⅜	13½
8	4½	1½	5	225	7 9/16	13½
10	5	1½	6	300	9⅜	14

NOTE.—"Maximum Economical Capacity" is based on 60 per cent. submergence and discharge of 12 gal. of water per min. per sq. in. area of discharge pipe in the smaller diams., and 15 gal. per min. per sq. in. in the larger sizes.

Example. Given a well 250 ft. deep, water 60 ft. below ground surface, but falls 25 ft. during pumping at rate of 200 gpm.; it is required to raise water 25 ft. above ground. Lift = 110 ft. If for lift of 110 ft. best percentage of submergence is 58, then submergence = (110 × 58) ÷ (100 − 58) = 152 ft. Quantity of free air per gal. = 110 ÷ 233 × log [(152 + 34) ÷ 34] = 0.59 cu. ft. (piston displacement). For 200 gals. = 118 cu. ft. To find starting and working pressures: Submergence at beginning of operations = 152 + 25 = 177 ft. Starting pressure must be just sufficient to overcome that due to the column of water above the foot-piece (bottom of air pipe) = 177 × 0.434 = 77 lbs. per sq. in. gage pressure. 152 × 0.434 = 66 lbs. gage, working pressure.

Actual pumping level in a well can seldom be known in advance of a test. Lift and submergence are assumed from other experience and the piping installed accordingly. Adjustments are then made on test by raising or lowering the pipes in the well.

Five methods of piping wells are shown in Figs. 122 and 123. For determining pipe sizes and capacities in the Saunders system, allow for 25-ft. lift, 15 to 20 gals. of water to each sq. in. area of water pipe, and for lifts of 50 to 125 ft. allow 12 to 15 gals. per sq. in.

Table 60. Central Air Pipe Air Lift: Pipe Sizes and Capacities

Size of casing, in	Size of air pipe, in	Capacity, gal per min
3½	1¼	80 to 100
4	1½	100 to 150
5	2	150 to 250
6	2	275 to 375
8	2½	500 to 650
10	2½	775 to 1000

Back-blowing and Cleaning a Well with Air. The standard practice of well drillers is to equip gravel and sand wells with strainers designed to shut out sand from working barrels of deep well pumps. These strainers become clogged. By back-blowing, the output from such wells can be permanently increased. The correct strainer for wells of this class, pumped by air lift, is a perforated screen with openings of a suitable size to admit the fine material into the well, from which it can be pumped, and to hold back the coarser particles, so as to form a natural filter. The force available for getting water into a well is head, due to difference between static level in water strata outside and the pumping level in the well, minus friction due to strata and screen. Therefore, the more this friction can be reduced, the greater will be the flow, provided an abundance of water is available. "Back-blowing" can be applied to all wells. Top of well casing should be sealed; next, by closing discharge pipe while air lift is in operation, air will be forced through foot-piece and will drive

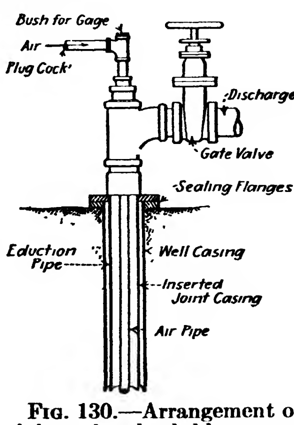


Fig. 130.—Arrangement of piping for back-blowing a well.

water ahead of it out through strainer and float the finer sand. Then, by opening discharge, flow will resume its course toward surface and bring a portion of floating sand with it. By a repetition of this operation and by increasing the back pressure if necessary, all fine sand immediately outside the strainer will be drawn into well and discharged at surface. When wells are drilled in rock, the action of the drilling tool forces cuttings back into crevices in the rock. These may be loosened and pumped out in the same manner. In cases of wells in fine material and quicksand it is often possible to set a strainer in the sand and drill auxiliary holes alongside the well down to top of strainer. Then foreign gravel may be dropped down, which will roll in alongside the screen and take the place of the sand pumped out—often increasing the yield fourfold, by affording outside of the gravel bed a larger area through which the water may leave the sand.

Booster System. By this means, Fig. 131, water may be lifted to an elevation above the surface by using again the air employed in the air lift proper. This system is especially suitable where the elevation must be accomplished

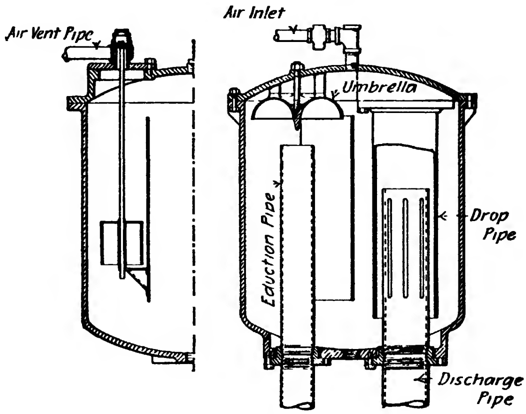


FIG. 131.

at some distance from the wells. The air goes to the top of the tank and escapes through a restricted vent, which maintains a back pressure upon water surface. The principle is as follows: Velocity of water and air in an air lift is always greatest as it nears discharge. By discharging into a closed tank, the kinetic energy of the column of water and air is used to recompress the air and a certain amount of the power expended is returned. Compressed air, under ordinary air-lift conditions, is wasteful, if used to force water horizontally, as the air goes to the top of the pipe and fails to bring the water with it. In the "booster" arrangement, the air does not follow the water, but converts its volume into pressure before being allowed to escape into the atmosphere, forcing a solid flow of water through the horizontal and vertical lines. The eduction pipe extends up into the tank, and the stream of air and water strikes a mushroom plate, which aids in the separation. The end of the discharge main also extends up into the chamber and has its end slotted; while a drop pipe, or sleeve, over this end compels the water to pass upward before entering the pipe. A baffleplate checks the splashing, and maintains comparatively

still water around the discharge. There is an open exhaust for the released air to escape to the atmosphere, but this is throttled by a valve so as to retain sufficient pressure above the water to force it through the discharge pipe to the required distance and elevation. With exhaust pipe full open it will discharge air and water; then the valve is partially closed until only air escapes. The engineman operates the complete plant from the compressor, no adjustment being required other than varying speed of compressor to secure a greater or less amount of water. Any number of wells and "boosters" may discharge into a common delivery pipe.

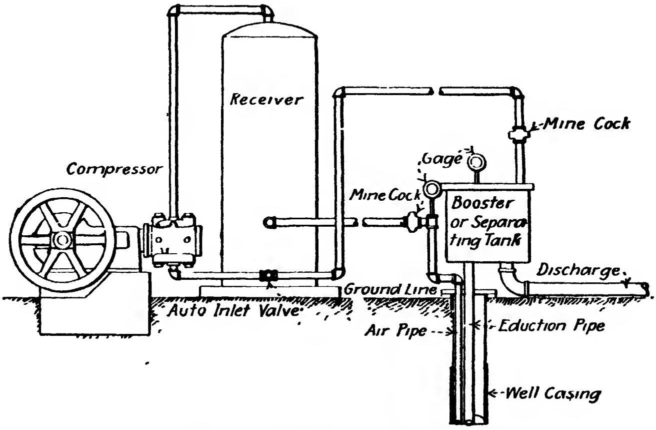


FIG. 132.—Sullivan air lift booster system for wells.

Return Air System. Another refinement is to return the air from the "boosters" to the intake of the compressor. An automatic inlet valve is placed on the return line near the compressor so as to supply from the atmosphere the air wasted. Advantages are: 1. Temperature of intake air has a direct effect upon volumetric efficiency of compressor, because, for every 5 degrees by which temperature of incoming air is reduced, there is a gain of one per cent. in volumetric efficiency. 2. By returning the air to the compressor under a slight pressure, the clearance losses are partially or entirely overcome. (Sullivan Machinery Co.)

Weber air-lift pumping system is based upon use of compressed air as piston. Lowest portion of well is cut off by plug and check-valve at bottom, and another plug above, with valves for passage of air and water. Compressed air forced into section of well between plugs forces water, which has come in through check-valve, upward through discharge pipe in solid column unmixed with air. When all water is forced out of this space, entrance is automatically stopped, and air under compression automatically goes over to fill portion of space in next well. Water rises in first well to take place of air, and this is forced out by second application of pressure, and so on. Plant in Long Island City, N. Y., is claimed to be pumping efficiently.

CHAPTER XI

INFILTRATION GALLERIES

Galleries vs. Wells. Infiltration galleries consist of trenches or tunnels excavated in water-bearing media below the water table. Under some circumstances, water can be brought to the surface by gravity. A gallery not only intercepts water more completely than wells, but it replaces the suction pipe, is more durable than either pipe or wells, and all trouble from pumping of air is avoided. On the other hand, wells draw deeper and more sterile waters, giving descending waters longer time to lose objectionable bacteria. Wells have higher depreciation, but less construction cost.

Construction.* In building an infiltration gallery, clayey material, if possible, is rammed solidly against the wall toward the valley and above the

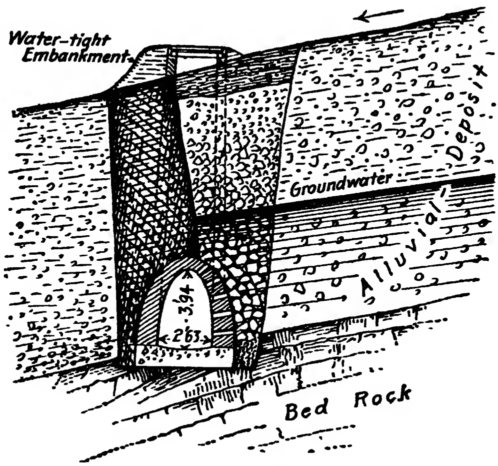


FIG. 133.
(König Wasserleitungen.)

roof so as to form an underground dam which backs up the underground stream, and forces it into the collecting gallery. Such a gallery collects as much water as possible from the ground, and the underground dam utilizes the ground as a collecting and equalizing reservoir. In refilling the trench on the side toward the hill, broken stone or gravel is solely employed, the coarser layers being next to the gallery; on the top just below the surface, a layer of impervious material is placed to prevent surface water from entering. Fig. 133 is a cross-section of a collecting gallery (large enough for a man to pass through) the bottom resting on the solid rock; the dotted lines indicate a manhole, which reaches 10 in. above the surface, to prevent earth, etc., from being washed in.

* König, Wasserleitungen und Wasserwerke, pp. 151–157.

According to size of plant, the inspection and ventilation shafts may be large enough for a man to enter, or only 8-in. pipes. Fig. 134 represents a collecting reservoir, in communication on both sides with two collecting galleries (large enough for a man to walk through); a delivery pipe leads from this reservoir, and the flow is regulated by a valve. The manhole may be vertically above the valve as indicated by broken lines in plan; this is recommended if water is at times to be stored in the shaft, as it is then possible to operate the valve from above. It often occurs, that for a given water supply, water from one

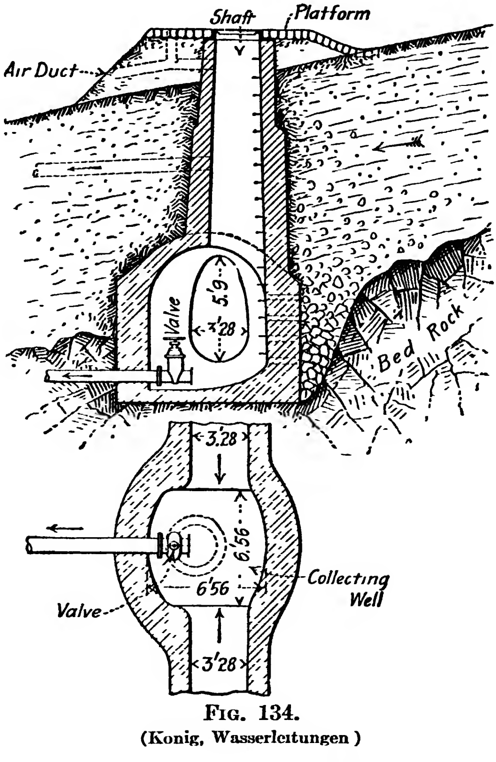


FIG. 134.
(Konig, Wasserleitungen)

valley is not sufficient and therefore waters from two or more valleys, at various levels, must be combined. As a rule, these waters from different valleys have not only different volumes, but also different heads, and the collecting gallery must be at such an elevation that water under the least head can flow away freely through it. So long as the combined inflow does not exceed the discharge, there is no possibility of a backflow into the various collection places; so soon, however, as inflow equals discharge, storage begins which extends upward in the collecting conduits, and into the ground-water passages. This impounding reaches especially into low-lying collecting spaces where, under such circumstances, stagnation ensues.

Overflows.* For the above reason it is necessary to provide an overflow in the collecting reservoir; its level must be so chosen with respect to the lowest

* König, Wasserleitungen und Wasserwerke, 1907.

area in which underground water collects, that water cannot rise in the latter above a certain level at times of high water. Further arrangements must be made so that water from the lowest collecting area first flows off and water from the collecting area next higher is drawn off only to such an extent as necessary to make up the total quantity required. The supply to the collecting reservoir from the different areas in which underground water collects is therefore regulated according to the levels of these different areas; regulation

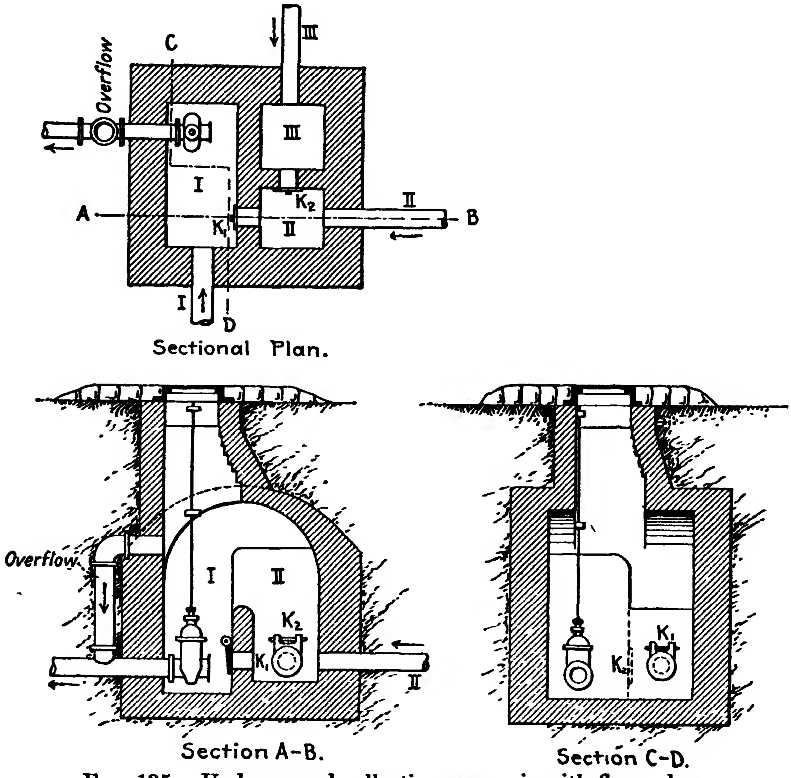


Fig. 135.—Underground collecting reservoir with flap valves.
(Konig, Wasserleitungen)

is accomplished by providing in the collecting reservoir one chamber each for water from each underground collecting area; each chamber communicates with the next through an automatic valve. These valves do not open until the water in the next chamber has reached its lowest level. From the lowest collecting area, the water is supplied directly to Chamber I, which also contains discharge pipe and overflow. Water from the area next higher first passes into Chamber II; thence it begins to flow into Chamber I through valve K_1 only when the draft through the discharge pipe becomes greater than the supply to Chamber I from its collecting area. Water from Area III first flows into Chamber III, and not until the draft through the discharge becomes greater than the supply from I and II does valve K_2 open to let water pass from Chamber III into Chamber II. These valves conserve the

water from the higher collecting areas and store it while the supply from the lower areas is sufficient. An overflow pipe communicating with the discharge prevents storage above a predetermined level.

Wantagh and Massapequa galleries of the Brooklyn, N. Y., water supply, respectively, 12,600 and 18,200 ft. long, consist of vitrified sewer pipe laid with open joints in sheeted trenches 10 to 15 ft. below normal ground-water level and surrounded with coarse gravel, over which a layer of fine gravel was placed. Sand was used for refilling the trenches. The whole region is a sandy, gravelly morainic outwash. The lower line of sheeting was usually left in place. Beginning at the pumping station at the central point, each gallery was excavated in both directions, approximately at right angles to the direction of underground flow. Gradients of the galleries toward their central wells are sufficient to carry double the estimated normal yield of 1,000,000 gal. daily per 1000 ft. Pipe diameters increase from 20 in. at extremities to 36 in. at the pump wells. To facilitate construction and to collect sand which may enter

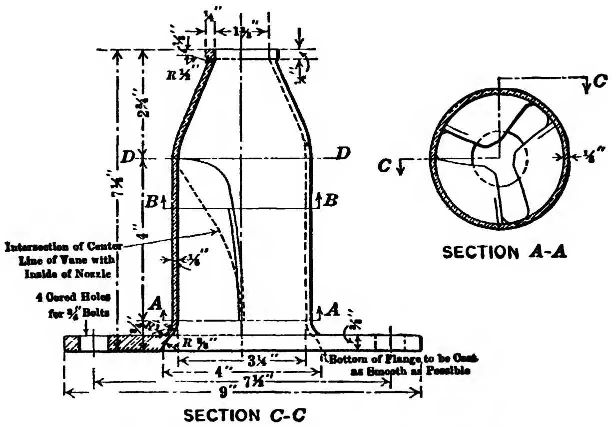


FIG. 135A.—Aeration nozzle, p. 260.

the galleries, manholes with sumps are provided every 250 ft. Crossing a village, Wantagh gallery was laid with cast-iron pipe for 1800 ft. to exclude polluted water. Construction cost proved to be about twice that for driven well systems in same locality, per million gallons daily, but total cost of water, delivered, including interest, sinking fund, taxes, operating expenses, extraordinary repairs and depreciation, is much less per mil. gal. than that of water obtained from the wells on the same watershed. (Wm. W. Brush.)

Examples and References. Golden, Col., E. N., June, 1891, p. 610. Newton, Mass., E. R., Nov., 1891, p. 418. Munich, E. R., June, 1898, p. 78. Daggett, Cal., E. N., Sept. 3, 1896, p. 157. Los Angeles, Cal., E. N., May 31, 1906. Naples, Italy, Proc. Inst. C. E., Vol. 84, 1885, p. 468. Dresden, E. R., Dec. 30, 1899, p. 722. Pueblo, Col., E. N., Jan. 17, 1891, p. 53. Beaver Falls, Pa., E. N., Aug. 20, 1896, p. 116. Owensboro, Ky., E. R., Aug. 31, 1907. Amsterdam, Holland, E. N., Apr. 27, 1905. Allegheny, Pa., E. N., May 17, 1900. Des Moines, Ia., E. R., Apr. 27, 1912. There are also plants at Zanesville, O., Parkersburg, W. Va., Newark, O., Gallipolis, O., and Ironton, O. (E. R., Feb. 4, 1911). (See also p. 655.)

CHAPTER XII

NOTES ON SOME EQUIPMENT FOR TREATING WATER

Nozzles for Aeration. Tests on models by New York Board of Water Supply, 1908 and 1910, showed practicability of casting thin bronze shell with spiral vanes in one piece. Shape finally selected for aerating Catskill water below Ashokan and Kensico dams is made up of cylindrical base surmounted by conical tip. For first installation tip opening will be $1\frac{3}{8}$ in. diameter. In base are cast 3 vanes, equally spaced on circumference, and projecting nearly to center of water-way. Angle between vanes and axis of nozzle varies from parallelism at base to 60° at top. Function of vanes is to set up rotary motion in jet, accelerating breaking up of jet into drops. Jet, 15 to 25 ft. high, gives effective aeration. Tests showed that this rotary motion gave jets greater resistance to distortion by wind, as compared to jets from vaneless nozzles. Tests on $1\frac{3}{8}$-in. bronze nozzles (Nov., 1910), with long tip and with short tip (also base was tested alone), using heads between 10 and 21 ft: Coefficient for discharge for base only (3 in. diam.) was 0.58; short tip, 0.89 to 0.99 ($1\frac{3}{8}$ in. diam.); long tip, 0.99 ($1\frac{3}{8}$ in. diam.). Quantity discharged varied from 0.28 to 1.79 c.f.s. Fig. 135A shows nozzle adopted.*

An aerator box, of steel, 9 ft. 3 in. by 6 ft. 6 in., and 4 ft. high, built by Harry W. Clark for the waterworks of South Norwalk, Conn., has 6836 holes $\frac{3}{16}$ in. diameter, spaced 1 in. between centers, in its bottom. This form was adopted after experiments in regard to quickest and most thorough method of introducing air into water free from air; that is, water which, owing to presence of much organic matter in state of change, had exhausted the free oxygen normally present. It was found that even weak sewage devoid of oxygen could be nearly saturated when dropping from such a box through $3\frac{1}{2}$ ft. of air. The small streams twist and break, and the water becomes a white, foaming mass about 6 in. below the box. Regulation is by float valve.†

Chemical Treatment. *Effect of hypochlorite of lime upon construction materials.*—Strong solutions of hypochlorite of lime, or bleach, rapidly corrode iron, steel and copper, but brass, bronze and tin resist well. Most woods are quickly destroyed, if unprotected. Portland cement mortar, slate crockery, paraffin, hard rubber, lead and galvanized iron are but slightly attacked, if at all.

Solution Tanks.—For the Anheuser-Busch brewery, St. Louis, coagulation plant, sulphate of alumina is dissolved in a concrete tank having three rectangular divisions each 10 by 7 by 5 ft. deep, in each of which from 500 to 2000 lb. can be dissolved in 2 to 5 hrs. Lime is slaked in iron tanks 12 by 12 by 7 ft. deep with sloping bottoms; each has also a perforated false bottom at a depth of 30 in., on which the lime is placed and partly submerged in 1 ft. of water; after slaking, which requires about 1 hr., the mixture is stirred and readily passes the perforations.‡

* See p. 259; also p. 690. † (Jl. N. E. W. W. A., March, 1916.)
‡ (T. A. S. C. E., Vol. 77, 1914, p. 1053, Edw. Flad.)

PART III
TRANSPORTATION AND DELIVERY OF WATER

CHAPTER XIII

OPEN CHANNELS

Forms of Section. For semicircular section running full, or for lower half of any regular polygon, also running full, R equals half radius of inscribed circle. Any such half regular polygon has minimum wetted perimeter for given area and consequently is of most advantageous form from a theoretical point of view; *i.e.*, to deliver maximum quantity of water per sec., Q, for given slope of bed, given area of water prism, and given number of sides for polygon. Of all trapezoidal sections running full and having a common side slope, or angle θ from the horizontal, that one is of most advantageous form whose three sides forming the wetted perimeter are tangent to the semicircle having a radius equal to the depth and with its center in the surface of the water; its hydraulic radius, R, is equal to half-depth. According to Prof. Bovey (Hydraulics, 2nd ed., p. 231), θ should not be greater than 63° 36′ for retaining walls; 45° for stiff earthen sides, faced; 33° 41′ for stiff earthen sides, unfaced; 26° 34′ for sides in light or sandy soil. To avoid erosion, velocities in some soils may have to be limited to 2 ft. per sec. or less. (Church, Hydraulic Motors 1908.)

Side slopes in channels and canals are chiefly dependent upon the nature of the materials through which dug, on climate and on velocity of the water. Suez Canal, Egypt: in sand, hard clay and rock, 1 on 2 (except in certain rock, where slope is 1 on 1); in clay, 1 on 3 to 1 on 3.5; in muddy sand and mud, 1 on 4 to 1 on 5. Vessels are allowed a speed of 6 mi. per hr.; the current is frequently 2 ft. per sec. St. Mary's Falls canal, Mich. (Sault Ste. Marie): in sand, 1 on 2; in clay, 1 on 1; rock excavated in the dry, vertical; rock excavated in the wet, 3 on 1 to 1 on 1; speed allowed, 9 mi. per hr. North Sea Canal, 1 on 2. Canal from Gand to Tiruauzin, Belgium and Holland, 1 on 3. Bruges canal, Belgium, 1 on 3. Lower Loire canal, France, 1 on 3. Cronstadt canal, Russia, 1 on 2. Walchiren canal, Holland and Belgium, 1 on 3.7. St. Louis canal, France (mixture of firm sand and clay, or natural earth), 1 on 2. Amsterdam canal, Holland, 1 on 2. Tunis canal, 1 on 2. Bizerta canal, 1 on 2.5. Kaiser Wilhelm canal, lower part, 1 on 3; upper part, 1 on 2. Havre canal, France, 1 on 2.

A slope of 1 on 1.5 should be safe in stable material, when lined with concrete, unless there be a probability of the canal being emptied, with saturated material back of the lining. Where freezing may occur, the upper part of lining should be heavier and backed with free-draining material, such as gravel or broken stone. In milder climates thin linings have been used successfully.

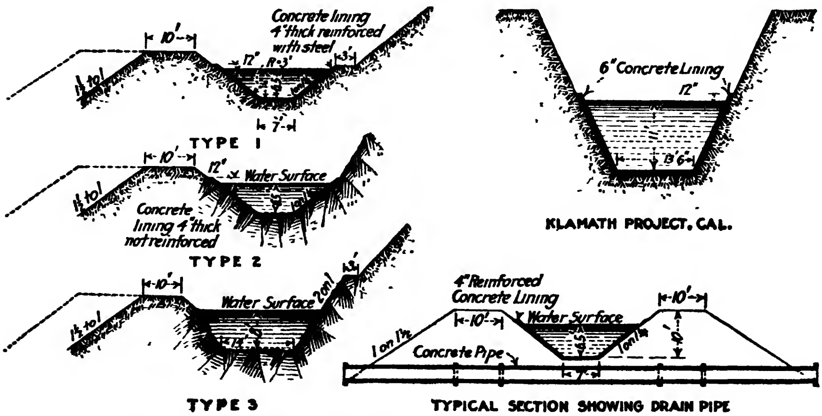


FIG. 136.—Open channels. U. S. Reclamation service.

FORMULAS FOR FLOW IN DITCHES AND CANALS*

Table 61. Safe Velocities in Unlined Earth Ditches
(W. P. Burr, E. N., Feb. 8, 1894)

Kind of earth	Velocity	
	Ft per sec	Ft per min
Soft brown earth	0 3	18
Soft loam	0 6	36
Pure sand.	1 1	66
Gravel	2 6	156
Sandy soil, 15 per cent. clay	1 2	72
Sandy soil, 40 per cent clay	1 8	108
Loamy soil, 65 per cent. clay	3.0	180
Clay loam, 85 per cent. clay	4 8	288
Agricultural clay, 95 per cent. clay	6 2	372
Clay	7 2	432

Caution in Use. For the practical application of any formula to an important new case, the engineer should always consult actual gagings, approximating as nearly as possible to the new case, whenever this can reasonably be done.

Mean Depth or Mean Radius.† The mean depth of water is expressed usually as "mean depth" for rivers, and as "mean radius" for artificial channels. There is a difference, sometimes of moment, between these two expressions. Applications of formulas, therefore, should take due account of it.

* See also Chapter XXV, page 551

† Rudolph Hering: Trans. Am Soc. C. E, 1911, Vol. 74, p 461.

Comparison of Mean Depth and Mean Radius

Rectangle:

$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{4} = 3.0$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12}{10} = 1.2$$

$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{6} = 2.0$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12}{10} = 1.2$$

Circle:

$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12.0}{3.91} = 3.07$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12}{12.3} = 0.98$$

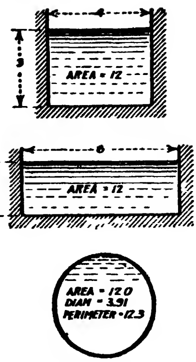


FIG. 137.

Johnston-Goodrich Formula. Investigations and measurements of the velocities in a number of unlined canals and ditches which varied widely as to material, grade, condition of repair, etc., were made in three principal groups—in earth, loose rock and solid rock; each group was subdivided into classes, which depended on alinement, maintenance, etc. From these it was deduced that not only does the value of coefficient C increase with an increase in hydraulic radius, R, but at the same time n decreases, producing a larger change in coefficient C than given by the usual formula. Diagrams (Fig. 138) are given, based on Kutter's formula, from which velocities can be obtained for different values of n and R, for slopes varying from 0.08 to 6 ft. in 1000. These diagrams have been used several years, and seem to have given satisfaction. In the exponential formula, $V = CR^PS^q$, $q = 0.5$ gave very close results; four different values of P, from 0.76 to 0.85, were found to satisfy the required conditions very well. For large values of R and velocities of 6 to 8 ft. per sec. or more, differences in the results may amount to 0.2 or 0.3 ft. per sec. (E. R., Nov. 4, 1911.)

Table 62. Exponents in Johnston-Goodrich Formula for Velocity in Ditches and Canals

Class	Condition	*C*	P
A	Clean straight ditch in earth or firm gravel, free from vegetable growth, made smooth by use.	59.8	0.76
B	Solid rock excavation, straight channel, rough face.	50.5	0.83
C	Solid rock excavation, crooked channel, rough face.	48.5	0.80
D	New ditch in earth, with straight channel.	45.5	0.80
E	Old ditch in earth, with straight channel and some vegetable growth.	42.8	0.83
F	New ditch in loose rock with straight channel.	41.5	0.83
G	New ditch in loose rock with crooked channel.	41.5	0.80
H	Old ditch in earth, with crooked channel, and some vegetable growth.	35.0	0.85
I	Old ditch in loose rock with crooked channel and some vegetable growth.	33.0	0.85

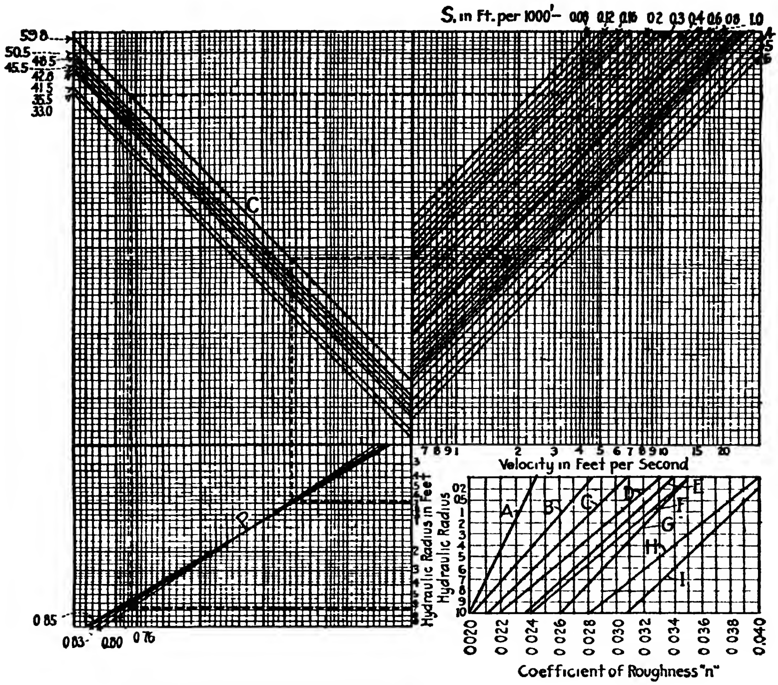


Fig. 138.

Kutter's Formula for Uniform Motion. (Same formula may be used for pipes and conduits.)

$$V = \left[\frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.6 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

R = hydraulic radius. S = slope. V = velocity.

n = 0 009, for well-planed timber evenly laid.

0 010, plaster in pure cement; glazed surfaces in good order.

0.011, plaster in cement with one-third sand; iron and cement pipes in good order and well laid.

0.012, unplaned timber, evenly laid and continuous.

0.013, ashlar masonry and well-laid brickwork; also the above when not in good condition nor well laid.

0.015, "canvas lining on frames;" brickwork of rough surface; foul iron pipes; badly jointed cement pipes.

0.017, rubble in plaster or cement in good order; inferior brickwork; tuberculated iron pipes; very fine and rammed gravel.

0.020, canals in very firm gravel; rubble in inferior condition; earth of even surface.

0.025, canals and rivers in perfect order and regimen and perfectly free from stones and weeds.

0.030, canals and rivers in earth in moderately good order and regimen, having stones and weeds occasionally.

0 035, canals and rivers in bad order and regimen, overgrown with vegetation, and strewn with stones and detritus.

Kutter's Formula for Rock Channels. In 1908, J. C. Stevens (E. N., June 8, 1911) made rating of Umatilla river, Ore., where flow is over water-worn basalt rock, free from silt, sharp corners and jagged edges. Values in Table 63 were derived from rating curve. Stevens suggests no less a value than n = 0.035 for unlined canals in newly excavated rock.

Table 63. Gaging of Flow in Water-worn Rock Channel

Gage hight, ft	Area, sq ft.	Mean vel., ft. per sec	Discharge, c f s	Slope	Hyd radius, R ft,	Chezy, C	Kutter, n
4	325	4 24	1,380	0 00435	1 87	47 0	0 0340
5	505	5 86	2,960	0 00415	2 77	54 7	0 0323
6	688	7 14	4,910	0 00395	3 70	59 0	0 0316
7	875	8 35	7,300	0 00375	4 58	63 8	0 0305
8	1,065	9 58	10,200	0 00355	5 46	68 8	0 0294
9	1,255	10 43	13,100	0 00335	6 32	71 7	0 0285

Chezy Formula applies to both pipes and open channels. $V = C\sqrt{R}\sqrt{S}$. V = velocity, ft. per sec.; R = hydraulic radius, ft.; S = slope = sine of angle water surface makes with the horizon.

$$C = \frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.6 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}.$$

Values of C for various values of Kutter's n, R and S are given in Trautwine's Engineers' Pocketbook. Kutter's expression for C is empirical and should be used with caution, keeping within limits based upon reliable data.

Wm. E. Foss Formula.*

$V^{\frac{11}{6}} = C_f R^{\frac{4}{3}} s$ for surfaces corresponding to Kutter's n = 0.009 to 0.017.
$V^2 = C_f R^{\frac{4}{3}} s$ for surfaces corresponding to Kutter's n = 0.020 to 0.035.
C_f varies with roughness; s = sine of inclination.

Table 64. Comparison of Kutter's n and Foss' C_f

Kutter's n	C_f	Kutter's n	C_f
0 009	23,000	0 017	6,000
0 010	19,000	0.020	5,000
0 011	15,000	0 025	3,000
0 012	12,000	0 030	2,000
0 013	10,000	0 035	1,500
0 015	8,000		

* See p 555 for pipes.

Table 65. Gagings in a Concrete-lined Channel, Umatilla Project, U. S. Reclamation Service

Hydraulic elements	Sections C	D	E	F	All
Length, ft.. ..	640	120	220	1075	2055
Slope.	0 0014	0 0017	0 0027	0.0017	0 0018
Average depth of water, ft	4 00	4 10	4 02	3 96	3 97
Mean area, sq. ft.	28 95	29 90	29 10	28 50	28 64
Mean hydraulic radius, ft. . ..	2 13	2 18	2 17	2 11	2 13
Quantity, cu. ft. per sec.. . .	----	----	----	----	205
Velocity, ft. per sec. . .	7.1	6 8	6 9	7 1	7 2
Chezy's C (page 265).	129	114	90	119	117
Kutter's n.... .	0 0132	0.0149	0.0189	0 0142	0 0146

The channel section is semicircular, 9.8 ft. diam.; ultimate capacity, 300 cu. ft. per sec. The four sections tested make up the main canal, entry and exit curves being omitted. Sections D and E are on a curve of 50-ft. radius, which caused great disturbance of flow. Sharp curvature cannot be introduced into canal alinement at such frequent intervals as will lead to the overlapping of the backing-up effect, thereby inducing an actual net loss of head. E. G. Hopson, E. R., Oct. 21, 1911, p. 480.

Surging in Open Channels. Surges in Tieton canal (8 ft. 3 in. diam., semicircular, concrete lined) seemed to obey no definite hydraulic law. With a maximum flow of 274 cu. ft. per sec., on some 75-ft. stretches water would be 3 ft. deep; on other similar tangents, 5 ft. deep. E. G. Hopson thinks that the canal was constructed with too rigorous a contraction at the headworks; at certain points variations in depth and velocity could not be explained by alinement or quality of the work. Points of turmoil occurred at about 200-ft. intervals, regardless of curve or tangents, being somewhat more noticeable on tangents. In addition there were traveling surges, 6 ft. higher than the surrounding water, which worked slowly downstream, beginning at a point where the depth was 4 ft., with quantity of 200 cu. ft. per sec. On curves, the water often heaped on the inside edge. The velocity was high enough to start a 25-lb. stone downstream. A velocity of 11 ft. per sec. caused waves of considerable size on uniform slopes. J. S. Conway believes that points of turmoil are generally caused by small irregularities in alinement and grade and traveling surges by loss and recovery of hydraulic equilibrium. H. F. Dunham believes that irregularities in the water surface may be to some extent explained by Spencer's Law of Rhythm. (Trans. Am. Soc. C. E., Vol. 71, 1911, p. 179.)

Curve Compensation in Open Channels. Markmann formula:

$$S_c = S + V^3 \div 2gR_c^2$$

S_c = tangent of slope in curve.
S = tangent of slope in straight channel.
V is the velocity in the straight channel, ft. per sec.
R_c = radius of curve, ft.

Eng.-Contr., Sept. 13 and Oct. 25, 1911, pp. 285 and 440.

Allowances for other losses of head, due to changes in section, admission at sides of streams with different angles and velocities of entrance, changes in quantities, etc., should be made as follows:

Change in section = between $\frac{(V_1 - V_2)^2}{2g}$ and zero, according to design.
Entrance of stream at side = between $V^2/2g$ and zero, according to design.
Change in quantity = $\frac{V_1^2}{2g} - \frac{V_2^2}{2g}$.
Starting loss = $V^2/2g$.
Loss due to entrance at upper end of canal = O to $\frac{1}{2}\frac{V^2}{2g}$.

Table 66. Flow of Water in Open Channels (Rectangular Wooden Flumes)*

Values of A, R, and $75A\sqrt{R}$ in formula $Q = 75A\sqrt{R}\sqrt{S}$

Size of flume, ft.	Safe depth of water, ft.	A	R†	$75A\sqrt{R}$
2 × 1	0.88	1.76	0.4681	90
2 × 2	1.75	3.50	0.6364	209
3 × 2	1.75	5.25	0.8077	354
3 × 3	2.62	7.86	0.9539	578
4 × 3	2.62	10.48	1.1342	841
4 × 4	3.50	14.00	1.2727	1187
5 × 4	3.50	17.50	1.4583	1590
6 × 4	3.50	21.00	1.6154	2001
6 × 5	4.50	27.00	1.8000	2713
7 × 5	4.50	31.50	1.9688	3307
8 × 5	4.50	36.00	2.1176	3915
8 × 6	5.50	44.00	2.3158	5016
9 × 6	5.50	49.50	2.4750	5829
10 × 6	5.50	55.00	2.6190	6683

Table 67. Flow of Water in Open Channels (Ditches in Earth)*

Side Slopes: 3 horiz to 4 vert   Values of A, R, and $50A\sqrt{R}$ in formula $Q = 50A\sqrt{R}\sqrt{S}$

Width at top, ft.	Width at bottom, ft.	Depth of ditch, ft.	Safe depth of water, ft.	A	R†	$50A\sqrt{R}$
2.4	1.0	0.9	0.8	1.28	0.4267	42
3.6	1.5	1.4	1.2	2.88	0.6400	115
4.7	2.0	1.8	1.6	5.12	0.8533	238
6.0	2.5	2.3	2.0	8.00	1.0667	412
7.0	3.0	2.7	2.4	11.52	1.2800	651
8.3	3.5	3.2	2.8	15.68	1.4933	957
9.4	4.0	3.6	3.2	20.48	1.7067	1,341
10.5	4.5	4.0	3.6	25.92	1.9200	1,801
11.8	5.0	4.5	4.0	32.00	2.1333	2,336
14.2	6.0	5.5	4.8	46.09	2.5606	3,687
16.6	7.0	6.4	5.6	62.72	2.9867	5,425
18.8	8.0	7.2	6.4	81.92	3.4133	7,577
21.0	9.0	8.0	7.2	103.68	3.8400	10,161
23.2	10.0	8.8	8.0	128.00	4.2667	13,248

* Pacific Coast Pipe Co.   † Based on safe depth.

Regulating Dams. Dams in the Wachusett open channel (Metropolitan W. W., Boston) for regulating velocity and providing for drawing water from beneath ice have been satisfactory. With low velocities (less than 4 ft. per sec.) gravel affords sufficient protection for the banks, but shoving of ice mars the water lines so that for appearance they need to be restored each spring. Depths of water flowing over such dams should be kept low; this frequently necessitates length of crest greater than width of channel; by building the dam in form of arc or V, extending upstream or downstream, any desired crest length can be obtained. Sluiceways controlled by gates, below ice level, make conveniently possible the drawing of water without disturbing the ice sheet or causing ice jams.

Table 68. Lining of Ditches and Reservoirs to Prevent Seepage

Experiments by Mead and Etcheverry (1906)

Description of lining	Average mean sinkage per hr in ft excluding evaporation	Efficiency ratio	Per cent. saving	Cost of experimental lining per sq ft	Cost of actual lining per sq. ft
Cement concrete, 3 in. thick	0 0046	7.17	86.6	$0.083	$0.075
Cement lime concrete, 3 in. thick	0.0114	2.90	65.5	0.083	0.075
Cement mortar .	0 0121	2 73	63.3	0 039	0 0325–0.035
Heavy oil (3 66 gal. per sq yd.)	0 0176	2.02	50.4	0.012	0 012
Clay puddle, 3½ in. thick	0 0185	1.78	47.8	0.039	0 012
Heavy oil (3 gal. per sq. yd.)	0 0220	1 50	38 0	0 010	0 010
Heavy oil (2 33 gal. per sq yd.)	0 0239	1.37	27.3	0.008	0 008
Thin oil (2½ gal. per sq. yd)	0 0329	1 08	7.3	0 01	0 008
Earth, unlined	0 0329 0 0355 0 0330	1 00	0.0		

Where water is valuable, concrete lining is more economical, besides being most durable. If water is plentiful, expense of concrete lining may not be justified; oiling will suffice. For some conditions pipes are preferable, to avoid seepage and evaporation. Very thin mortar or concrete linings cannot be used where frost is destructive. (Agri. Experiment Sta., Berkeley, Cal., 1907, Bulletin 188.)

FLUMES

Wooden flumes vary widely in size and in conditions of service, consequently in details of design. Yellow pine is one of the best materials. A recent successful design is shown in Fig. 139. To maintain tightness of joints and prevent decay of wood, flumes should be kept full of water, or at least thoroughly wet. If necessarily empty or partially empty at times, wooden flumes should be sheltered from hot sun and drying winds by an inexpensive roof and side sheathing.

Steel flumes, semicircular troughs of galvanized sheet steel (patented interlocking joints) supported by trestles of wood or structural steel, are made by several manufacturers in stock sizes. Information about those made by Hess Flume Co., Denver, Col. (Hess and Maginnis flumes) is here given as more or less typical.

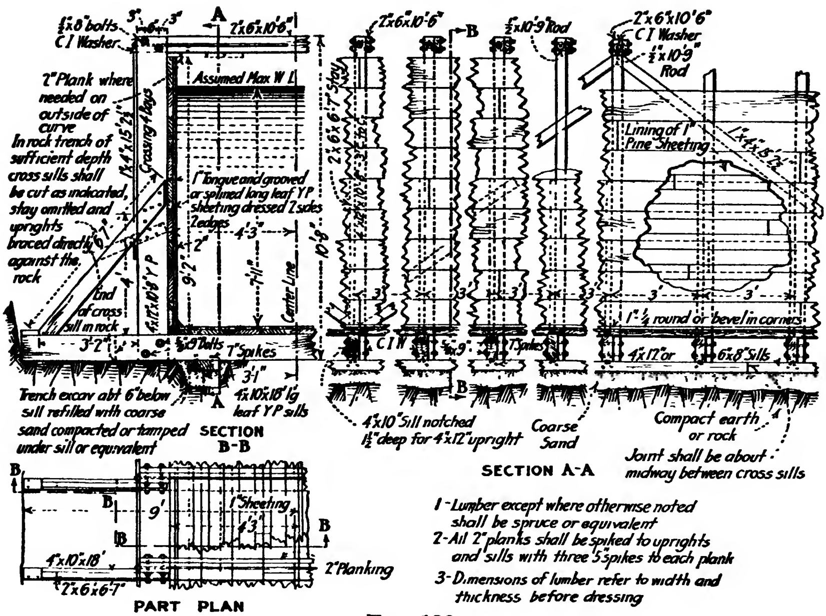


FIG. 139.

Table 69. Sizes of Timber Substructures for all Sizes of Hess Improved Metallic Flumes

(For hight of 12 ft and 16 ft. c. to c bents)

Flume No	Diameter, ft. in	Area, sq. ft.	Total wt. per foot complete (flume full of water), lbs	Size of substructure timbers, in: Posts	X braces	Knee braces	Stringers*	Dist. betw'n stringers, ft. in.	Carrier beams: Size, in †	Total length, ft. in.	Dist c.–c. of holes for carriers ft in	Ft. B M per linear foot of flume: Substructure	Carrier b'ms	Total
12	8	0 17	15	2 × 4	1 × 4	1 × 4	2 × 4	9½	a	1 2	8½	3 44	0 30	3 74
15	10	0 27	22	2 × 4	1 × 4	1 × 4	2 × 4	11½	a	1 4	10½	3 44	0 40	3 84
18	1 0	0 39	41	2 × 4	1 × 4	1 × 4	2 × 4	1 1½	b	1 6	1 0½	3 44	0 50	3 94
24	1 3½	0 62	55	4 × 4	2 × 4	2 × 4	2 × 6	1 5	b	1 10	1 3¾	7 1	0 60	7 70
30	1 8	1 09	80	4 × 4	2 × 4	2 × 4	2 × 6	1 10	c	2 4	1 8½	7 1	0 70	7 8
36	1 11	1 47	109	4 × 4	2 × 4	2 × 4	2 × 6	2 1	c	2 8	2 0½	7 1	0 86	7 96
42	2 3	1 98	135	4 × 4	2 × 6	2 × 6	2 × 6	2 5	d	3 0	2 3½	8.27	0.93	9 1
48	2 6½	2 54	171	4 × 4	2 × 6	2 × 6	2 × 6	2 9	d	3 4	2 7	8 27	1 00	9 27
60	3 2¾	3 97	247	4 × 4	2 × 6	2 × 6	2 × 6	3 5	d	4 0	3 3	8 35	1 14	9 42
72	3 9¾	5 73	387	6 × 6	2 × 6	3 × 6	3 × 8	4 0	e	4 8	3 10½	14 2	1 36	15 6
84	4 5¼	7.74	533	6 × 6	2 × 6	3 × 6	3 × 8	4 8	e	5 6	4 6	14 3	2 35	16 6
96	5 1	10 13	683	6 × 6	2 × 6	3 × 6	3 × 8	5 4	e	6 0	5 1¾	14 4	2 57	17 0
108	5 8¾	12 89	860	6 × 6	2 × 6	3 × 6	3 × 8	6 0	e	6 10	5 9½	14 5	2 93	17 4
120	6 4¾	15 88	1088	6 × 6	2 × 8	3 × 8	3 × 10	6 8	f	7 6	6 5¼	17 2	4 45	21 6
132	7 0	19 24	1303	6 × 6	2 × 8	3 × 8	3 × 10	7 4	f	8 0	7 0¾	17 3	4 74	22 0
144	7 7¾	22 92	1517	6 × 6	2 × 8	4 × 8	3 × 12	8 0	f	9 0	7 8¼	19 9	5 33	25 3
156	8 4	26 74	1783	8 × 8	3 × 8	4 × 8	3 × 12	8 8	f	9 6	8 5	27 3	5 63	32 9
168	8 11	31.08	2065	8 × 8	3 × 8	4 × 8	3 × 12	9 4	f	10 0	9 0	27 5	5 91	33 4
180	9 6¾	35 87	2369	8 × 8	3 × 8	4 × 8	3 × 12	10 0	f	11 0	9 8	27 7	6 52	34 2
192	10 2	40 50	2675	8 × 8	3 × 8	4 × 8	6 × 10	10 7	f	12 0	10 3¼	29 9	7 11	37 0
204	10 10	46.08	3012	10 × 10	3 × 10	4 × 10	6 × 10	11 3	f	13 0	10 11¼	37 7	11 53	49 2
216	11 6	51.90	3425	10 × 10	3 × 10	4 × 10	6 × 12	11 11	g	14 0	11 7¼	40 2	12 43	52 6
228	12 1	57 27	3745	10 × 10	3 × 12	4 × 12	6 × 12	12 6	g	15 0	12 2¾	42 9	13 36	56 3
240	12 9	63 71	4145	10 × 10	3 × 12	4 × 12	8 × 12	13 2	g	16 0	12 10½	48 0	14 20	62 2
252	13 5	70.47	4629	12 × 12	3 × 12	4 × 12	8 × 12	13 11	g	16 6	13 6¼	51.0	15 3	66 3

* Stringers run between bents, outside of flume, and support transverse beams (carriers) from which flume is suspended.

† a, 1 × 2; b, 2 × 2; c, 2 × 3; d, 2 × 4; e, 3 × 4; f, 4 × 4; g, 4 × 6.

More recent type of steel flume (E. N., Dec. 28, 1911) eliminates timber cross-ties, lateral support being provided by two parallel timbers at top of flume, to which metal is fastened by a longitudinal stiffening angle. Advantage lies in free channel, doing away with possible obstructions and consequent overflowings. Metal flumes are also made by Chicago Bridge and Iron Works, Graver Tank Works, and Minneapolis Steel and Machine Co. From hydraulic viewpoints, preference should be given to a type of flume free from ridges or projections in the waterway.

Reinforced Concrete Flumes. Canadian Pacific Ry. Co. built (E. N., Aug. 8, 1912, p. 247)* near Calgary, Alberta, reinforced concrete flume, 10,500 ft. long, with maximum hight 54 ft., cross-sectional area, 129 sq. ft. Section in form of hydrostatic catenary composed of reinforced concrete shell only 5 in. thick, suspended from reinforced concrete structure.

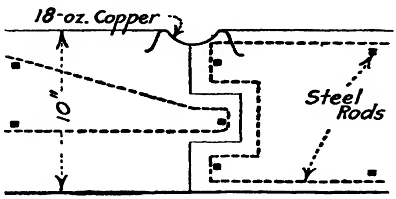


FIG. 140.

The reinforced concrete flume of the Northern Idaho and Montana Power Co., Big Fork, Mont. (10 by 22 ft. in cross-section) was open to the air. It was necessary to design the expansion joints (Fig. 140) water-tight as the freezing of water in the joints under a winter minimum of − 30° F. might cause spalling. A flexible copper covering was provided to permit movement as well as keep water out of the joint. An ordinary tongue-and-groove joint was used, reinforced to prevent breaking. The arc of copper was of sufficient length, so that at the lowest temperature, it would not be straight. The side of the copper next to the concrete was greased to prevent adhesion. The effectiveness is dependent on the adhesion of the ends of the copper to the concrete. Tests with 18-oz. copper inside a standard briquette, 1 : 2 mix, 28 days old, gave the following results:

Type	Pull to break concrete	Pull to break bond between copper and concrete
Without copper	240 lbs.	
Copper, cleaned	308 lbs.	364 lbs.
Copper, uncleaned	346 lbs.	431 lbs.

Approximately 2 sq. in. of copper was in contact; adhesion per sq. in. was 215 lbs. with uncleaned copper, and 182 with cleaned. (E. R., March 2, 1912.)

* Also E. N., Dec. 28, 1914.

CHAPTER XIV

AQUEDUCTS

CUT-AND-COVER AQUEDUCTS

Shape. Masonry aqueducts and similar conduits, unless reinforced with steel, are customarily built with approximately horse-shoe cross-section, on account of good hydraulic properties, resistance to earth loads, facility and economy in construction, and convenience in maintenance.

Inverts. Aqueduct inverts should be dished only enough for convenience in cleaning and draining unless dishing is required to form an arch to resist

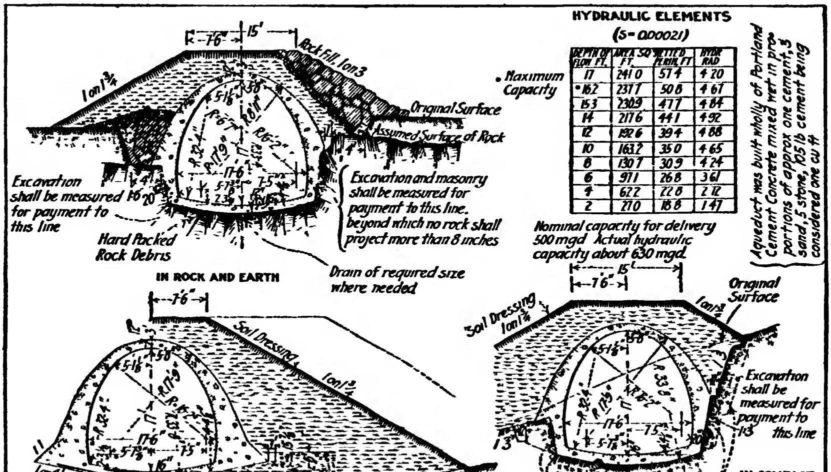


FIG. 141.—Catskill aqueduct. Standard types, open cut and embankment.

side thrust at bottom of walls from earth load or to take upward water pressure. In very large aqueducts it may be advisable to provide a level ledge on one side for a foot-way. In determining the thickness of inverts, allowance should be made for a few inches of concrete at the bottom, more or less mixed with dirt, which will have but little strength, especially if placed directly on earth not very hard. The Wachusett aqueduct, of horse-shoe cross-section, has a maximum width of 11 ft. 6 in. and hight of 10 ft. 6 in. The inside radius of the invert is 14 ft. In dry ground, the invert is 6 in.

of natural cement concrete, lined with one ring of brick on edge. In wet ground, grooved board platforms were placed in the bottom of the trench before placing concrete, and wherever the head of the ground water on the invert exceeded 6 ft., two rings of brick were used. In two or three places, where the upward pressure of the ground water was about 16 or 17 ft., and two rings of brick were used, the brickwork was lifted; these partial failures are thought to have been due in some measure to faulty work. The thicknesses of masonry in the Wachusett and Weston aqueducts are thought to be about the minimum desirable for aqueducts of such size, character and conditions.

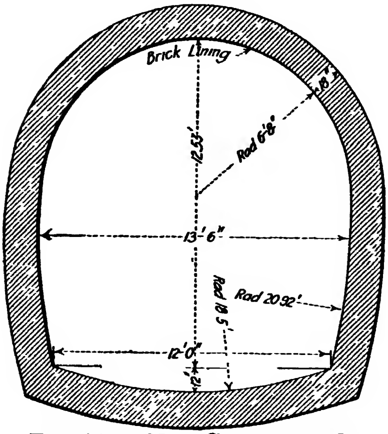


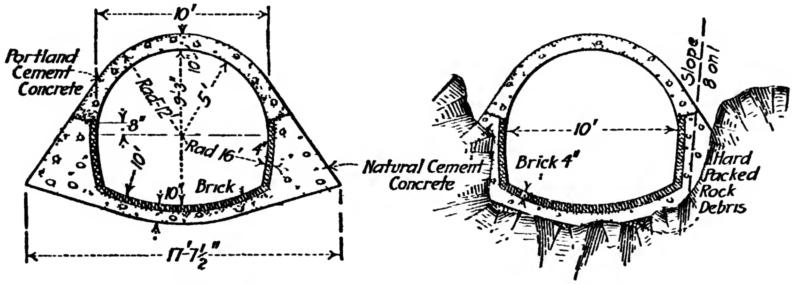
Fig. 142.—New Croton aqueduct. Standard type above Jerome Park reservoir.
(Except deep pressure tunnel at Gould's Swamp)

Alinement. Experience in the construction and operation of the Wachusett and Weston aqueducts (Boston) shows no practical objection to sharp curves, the radius depending somewhat on the size of aqueduct; a radius of 100 to 200 ft. can be used on large aqueducts. Curves should be standardized and limited in the number of radii, and forms should be well made, for convenience and economy.

Changes in Cross-section. Experience indicates no serious objections to changes in cross-section of an aqueduct, if made by transition pieces without sudden enlargement or contraction. Such changes are sometimes desirable to conform to steeper or flatter slopes, to secure economical construction in tunnels, or for depressed portions under pressure, as beneath railroads or highways, and in similar places.

Cracks. A. L. Johnson has advanced the theory that continuous concrete walls will crack vertically in sections such that the weight of the section multiplied by the coefficient of friction on the soil is equal to the tensile strength of the wall. There have been no longitudinal cracks in the arch of Wachusett and Weston aqueducts, but transverse cracks have appeared at dividing places between days' work. In Catskill aqueduct, transverse cracks have been observed in a few 75-ft. sections but none in 60-ft. sections. No cracks wider than 0.04 in. were found in the Wachusett aqueduct. In winter, the cracks were cut out by an expert mason with a diamond-pointed chisel, to a width of ¼ in., and a depth of 1¼ in. and pointed full with Portland cement mortar. Many parted again, but only so as to show an incipient crack at the surface. So far as observed, these secondary cracks do not go deeper than ¾ in., and no leakage took place. The aqueduct 4000 ft. long connecting Jerome Park reservoir, N. Y., with New Croton aqueduct, was constructed (1904) with no attention to temperature stresses; a few trivial transverse cracks have developed. The aqueduct is built of Portland cement concrete, horseshoe section. The trench is in firm earth and rock.

Expansion Joints. Chas. S. Gowen advocated a transverse joint every 40 ft. If the interval between expansion joints be more than 50 or 60 ft., cracks often develop between; a complete vertical transverse joint should be inserted at intervals of 50 to 75 ft., in a concrete aqueduct of any size. Grease, asphalt, paper, soft soap, etc., may be used to assure these joints. This method also allows for moderate settlement on embankments without injury. Some

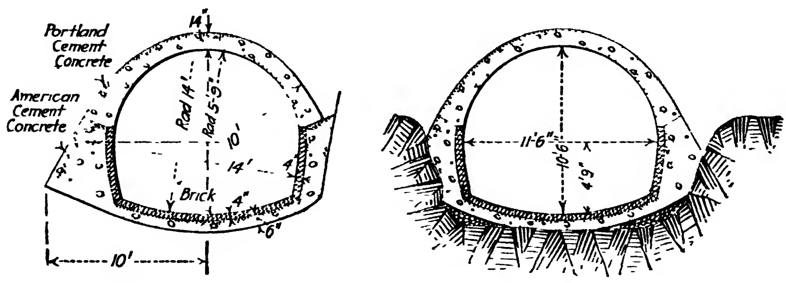


In Loose Earth — In Rock

FIG. 143.—Weston aqueduct.
(Gradient 1 on 1250)

engineers prefer to make no provision for expansion joints, but to treat such cracks as may occur. No expansion joints were provided in the Los Angeles aqueduct from Owens river. Expansion joints commonly require some treatment to prevent small but objectionable leakage.

Hydrostatic tests on the newly completed portions of Catskill aqueduct showed that under severe conditions considerable leakage may occur at the joints. Tongue-and-groove expansion joints were used almost exclusively at first. As the groove forms had to be pulled in about 16 hrs., due to the



In Loose Earth — In Compact Earth — In Rock

FIG. 144.—Wachusett (Nashua) aqueduct.

progress of the work, it was difficult to secure smooth sides and a true shape, which are essential if the tongue is to move freely in the groove. The small draw used in the tongue-and-groove joint, the relative narrowness of the tongue compared to depth, and the accidental skew of the bulkheads relative to the axis of the aqueduct, tend to cause a lock joint rather than a slip joint. Some tongues were found broken in cold weather. The consensus of opinion

was that the tongue-and-groove joint was not practical, although giving excellent results when carefully made. Hydrostatic tests showed that leakage may be expected through invert joints. Consequently, for water stops in joints

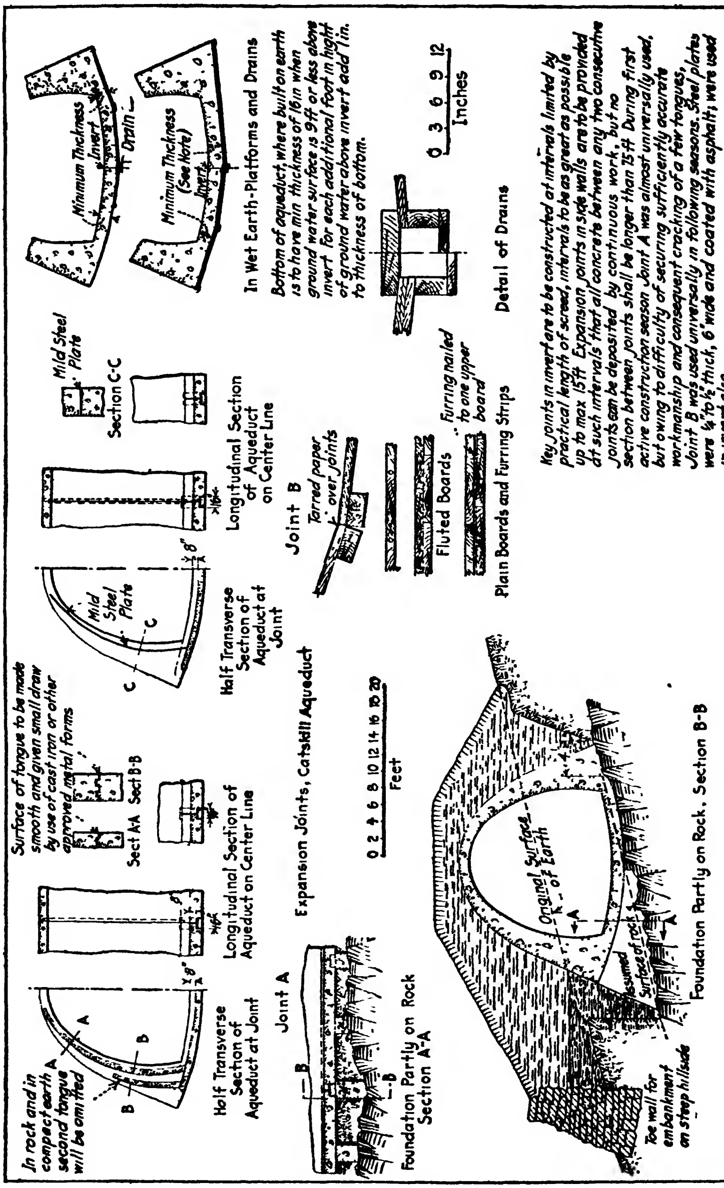


FIG. 145.—Catskill aqueduct. Standard details in open cut. Expansion joints and drains.

between days' work in arch and side walls, and between sections of invert, steel bands about ⅜ in. thick by 6 in. wide, dipped or painted with asphalt, were adopted. Half the width was buried in the concrete first cast. Joints

which opened were treated experimentally in many ways. The method adopted as simplest and most effective was to point the face of the crack with Portland cement mortar, and after this was hardened, to grout the space back of the pointing through a small pipe nipple, set near the bottom, using a small hand force pump (Douglas). In some cases, the joints were raked out. Grout was poured sometimes instead of being pumped.

Lead water stops have not proved wholly effective, and are not deemed worth the trouble and expense. An L-shaped joint smeared with asphalt, or some similar device, would probably be better than lead water stops.

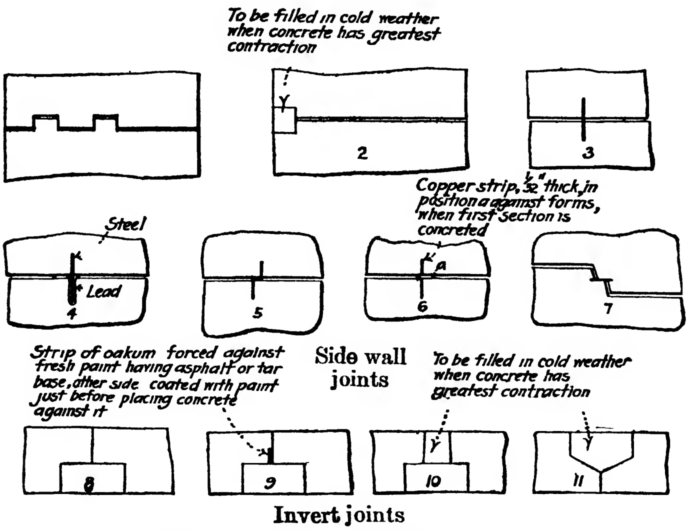


FIG. 146.—Expansion joints for cut-and-cover aqueducts.

The metal for joints 3 to 5 inclusive may be either mild steel or copper; steel is suitable for joints 3 and 4 where metal is comparatively thick and copper is better for joints using thin metal on account of its non-corrosive property and greater pliability. Lead, which has been suggested for some of these joints, does not offer sufficient resistance to damage and displacement while depositing concrete, if of practicable thickness.

Joints 3 to 7 are applicable to the invert if used alone or in combination with a concrete key.

All contact surfaces between new and old concrete at expansion joints to be coated with paint having asphalt or tar base.

Treating joints when open in cold weather gives the best results. Cement grout, lead wool, lead wire, hot tar, iron filings and sal ammoniac, oakum treated with pitch or asphaltum, or dipped in grout, all have been used. That leakage from masonry aqueducts cannot be wholly prevented is evident from universal experience. Although small quantities of silt, etc., carried in the water may reduce the leakage, even after many years some leakage still occurs, as can be seen by examining the older aqueducts. Careful construction can keep the leakage small. Expansion joints and cracks should have special treatment (pointing, calking, etc.) before the aqueduct is put in service, and subsequently, if necessary. Where the ground water is relatively high, the leakage may be inward.

The maximum opening for expansion joints will be determined by the range in temperature between the maximum reached during the placing of the concrete (95° ± F.) and the minimum subsequently reached (36° or lower).

Weekly readings of water at the inlet and at the 135th St. gatehouse of the New Croton aqueduct, 1913 and 1914, showed a maximum of 74° and a minimum of 33°. The extreme minimum in Catskill aqueduct is estimated at 36°, and the average minimum, 40°. The water temperature may reach 70° in summer, and will probably average 65°. Of course, local climatic conditions, size of conduit and depth of cover will affect these temperatures.

Forms. On Catskill aqueduct collapsible steel forms of Blaw, Ransome and other makes were employed. The tendency at first was to make the forms too light and with poorly fitting joints (see Fig. 141 for section of this aqueduct). The greatest horizontal movement in the inside forms occurred about 10 ft. above the invert, and in some cases amounted to 0.02 to 0.14 ft., depending on design. The maximum displacement at the top was about 0.15 ft., with an average of about 0.03 ft. The displacement depends directly on the head of fresh concrete. Several factors, aside from stiffness, affect the movement of forms: (1) character of the interior bracing; (2) type of aqueduct; in "firm earth" type, there is no support of the outside forms in the lower portion; (3) weather conditions; in cool weather, concrete sets slowly, increasing the hydrostatic pressure of liquid concrete; (4) the length of arch concreted and the number of batches of concrete placed per hr.; the more rapidly the forms are filled, the greater will be the pressure and deformation; (5) the number, size and position of stay-bolts between inside and outside forms. A light form, properly braced, gives a good aqueduct section, but will not stand handling.

The removal of the bulkhead of forms was expedited by some contractors on the Catskill aqueduct by the aid of a simple steam coil in the bulkhead, kept warm over night, thus accelerating the hardening of the concrete. Combined air and water jets under pressure were found very effective in cleaning forms. Steam jets hastened the drying of forms after cleaning. After the removal of all forms, all bolt holes through the concrete, if any, must be *thoroughly filled* with mortar; a little mortar plug will not do. See "Concrete forms for Catskill aqueduct," by Alfred D. Flinn, Jl. Am. Concrete Inst., June, 1915.

Interior Finish. A frequent trouble with concrete is peeling at the crown, as forms are removed, due to suction or adhesion, or to green concrete. On one aqueduct, Form oil gave poor results in hot weather; trouble was eliminated when the thin Form oil was replaced by a mixture of cup grease and black oil. Good results have been obtained by the use of crude vaseline and crude oil, Albany grease, and "Special Coating for Concrete Forms" (Manning, Maxwell and Moore). Thorough cleaning of forms is the most essential requirement for obtaining a good finish; no lubricant will prevent peeling and other defects in the surface, due to forms that are not clean. Peeling at the crown was prevented where the tops of forms were cleaned, dried and greased just before placing the last concrete at the top of the arch, and where care was taken in properly removing the forms. Forms hinged at the crown and so arranged that they could be separated from the concrete progressively from the bottoms of the side walls to the crown caused little injury to concrete surfaces. Porous concrete occurred at bottoms of side walls, by washing out of mortar through leaks in the forms.

Culverts. Culverts beneath large aqueducts should be of liberal size, none smaller than 4 sq. ft. area of waterway, preferably of concrete or other substantial masonry, and smooth inside. Experience indicates that culverts should not be fitted too closely to conditions existing when they are designed, since some culvert watersheds may later be increased in area by diversions, and others discharge more quickly because improved. In any region in which urban or suburban conditions may be brought about in even a remote future, culverts should be proportioned to care safely for very quick run-off. Place the invert of the culvert somewhat lower than the natural bed of the stream, to allow for future improvement of the stream which may lower the bed and increase the run-off. For flows of ordinary occurrence, a velocity of 6 ft. per sec. should not be exceeded, in designing culvert openings, but occasional floods of 10 ft. per sec. are allowed so long as the channel leading away from the culvert is wide enough to reduce this velocity properly. With substantial paving between the wings of the culvert, and a short distance beyond, there would be no trouble from erosion. With velocities greater than 10 ft., the head at entrance causes an undesirable backing up of water. By selecting from list of coefficients below values which suit the character of the country, an area can be determined safely and still not unreasonably large. Studies show that the Kuichling formula gives very large results compared with others for which coefficients suited to the Croton watershed are used. All formulas except Myer's give curves of the same general shape. The Talbot curve ($c = 0.3$) strikes a fair average between the U. S. Geological Survey, Burkli-Ziegler, and Fanning curves for $V = 8$, and other coefficients can be selected to average those for other velocities. It is well to make culverts of such size and shape that they can be cleaned readily.

Comparison of Run-off and Culvert Formulas

(*a*) *For Culvert Areas direct.* Talbot: $A = c\sqrt[4]{D_a^3}$. Myer: $A = c\sqrt{D_a}$

(*b*) *For Run-off from Drainage Area.* Burkli-Ziegler: $Q_a = cR\sqrt[4]{\dfrac{S}{D_a}}$

Kuichling (Mohawk Valley): $Q_m = \dfrac{44{,}000}{D_m + 370} + 20$

$$Q_m = \frac{127{,}000}{D_m + 370} + 7.4$$

United States Geol. Survey: $Q_m = \dfrac{46{,}790}{D_m + 320} + 15$ (see Water Supply & Irrigation Paper No. 147, 1905, for Eastern U. S.).

Fanning: $Q^* = 200(D_m)^{\frac{5}{6}}$. $\quad Q_m = \dfrac{200(D_m)^{\frac{5}{6}}}{D_m}$ (N. E. and Middle States Basins).

A = Area of waterway, in sq. ft.
D_a = Drainage area in acres.
D_m = Drainage area in sq. mi.
Q_a = Run-off in sec. ft. per acre.
Q_m = Run-off in sec. ft. per sq. mi.
S = Average slope of ground, ft. per 1000.
c = coefficient.
R = Rainfall in inches per hr.

* Q = run-off sec. ft.

Coefficients, c

Myer. Hilly ground, 1.5. Mountainous and rocky ground, 4.

Talbot. Rolling agricultural country subject to floods at times of melting snow, and with length of valley 3 or 4 times width, ⅓. Steep and rocky ground, ⅔ to 1.

Burkli-Ziegler. (For run-off into sewers.) Paved streets, 0.75. Ordinary cases, 0.625. Suburbs with gardens, lawns and macadamized streets, 0.31. (T. A. S. C. E., Vol. 10, 1881.)

Foundation Embankments. Embankments under Boston's aqueducts (Sudbury, 1878; Wachusett, 1898; Weston, 1903) have not settled perceptibly or so as to cause cracks. Slopes (1 on 1¾) of cover embankments have stood well. Weston aqueduct was built largely on embankment, thoroughly rolled in 4-in. layers of carefully selected materials. Banks stood 6 weeks or more before placing masonry. Some embankments are soaked by forming shallow pools on top when built to full hight. After such soaking, the embankment should be given ample time to drain free of surplus water and harden before building masonry thereon. The thickness of layers may be 3 in. to 6 in., according to materials, hight of embankment, etc. Each layer must be very solidly compacted. The embankment should be built 2 ft. or more above final elevation, and the surplus excavated just before the masonry is built.

Flotation. A mathematical examination of the empty Catskill aqueduct for buoyancy showed that if both ends of a portion were stopped, the standard section for dry loose earth (Fig. 141) would not float with water standing in trench 21 ft. above the invert, if backfilled; without earth cover, 13 ft. 4 in. of water would be required to float. Possibility of flotation should always be examined if any conduit is located so as to be largely or wholly below ground-water level or so as to be under water in floods.

Waste weirs are so placed in the Catskill aqueduct as to prevent more than 3 ft. head above the aqueduct arch, in event of improper operation of the gates.

Estimating Quantities, Catskill Aqueduct. Fig. 141 shows the principal types. For rapid estimates of quantities per 100 lin. ft., Figs. 147, 148 and 149 were used with topography. The established hydraulic gradient gives invert elevation at any station. If the section is all in earth, use Fig. 147 only. All principal curves are based on a horizontal surface. In case of sloping surface, add the quantity derived from the correction curves. Fig. 147 applies to aqueduct either in earth cut or on embankment; former condition requires no rolled embankment; the latter, no excavation. Often the aqueduct is part in cut and part on fill; for which case, obvious combination of these curves can be made. Figs. 148 and 149 are for aqueduct in rock and earth. Directions on Fig. 148 should be followed. Fig. 148 gives bottom width of earth excavation, which is necessary for use on Fig. 149. These diagrams are offered as an example of a method rather than for direct use.

AQUEDUCT BRIDGES

Cabin John Bridge, a stone arch of 220 ft. span, carrying Washington aqueduct over the run, was built in 1863. The conduit is 9 ft. in diam. Owing to the extension of the reservoir system, the crown of the conduit is now under

2 ft. head. The original depth of water being 7 ft., the upper segment of the arch was not plastered. Leakage became so great in 1905 that the remainder

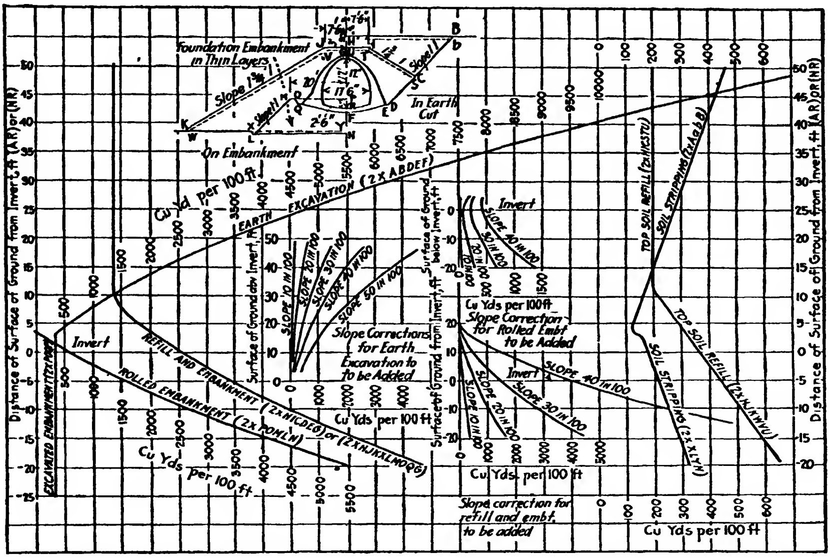


FIG. 147.—Earth quantities for cut-and-cover aqueduct (Fig. 141). Catskill aqueduct.

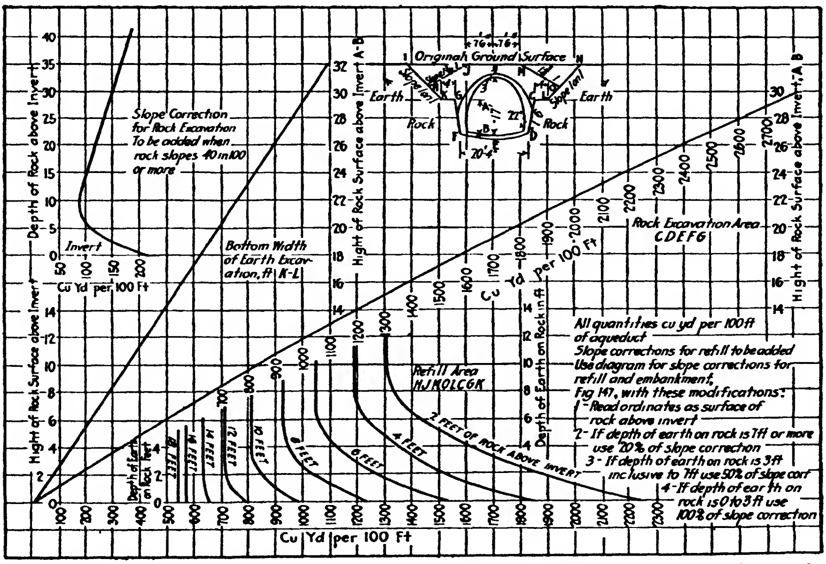


FIG. 148.—Quantities for cut-and-cover aqueduct in rock and earth (Fig. 145).

of the joints were pointed, remedying 90 per cent. of the leakage. The remaining leakage came from temperature cracks caused by the unequal expansion of the 2 sides, the axis of the bridge lying nearly due east and west. These

long temperature cracks lie in the lower quadrant of the conduit on north side of the bridge. The unequal expansion has opened the brick rings of the conduit along lengths of cracks, and the upper quadrant is moving over the lower at a rate of $\frac{1}{8}$ in. per yr. The remedy is a segmental cast-iron lining, 8 ft. 6 in. diam. (E. R., Apr. 1, 1911, and E. N., July 10, 1913.)

Assabet Bridge, Wachusett Aqueduct, has a length of 359 ft., composed of 7 granite arches (1 over road and 6 over a mill pond), with spans of 29.5 ft.; hight of the crowns above pond, 17 ft. The lower half of the aqueduct on the bridge has the same form as at other places, but the upper part has vertical walls, roofed with I-beams and brick arches, instead of the semicircular arch. A lead lining was provided to prevent leakage from the aqueduct where it crosses the bridge, and it was absolutely water tight, winter and summer, for

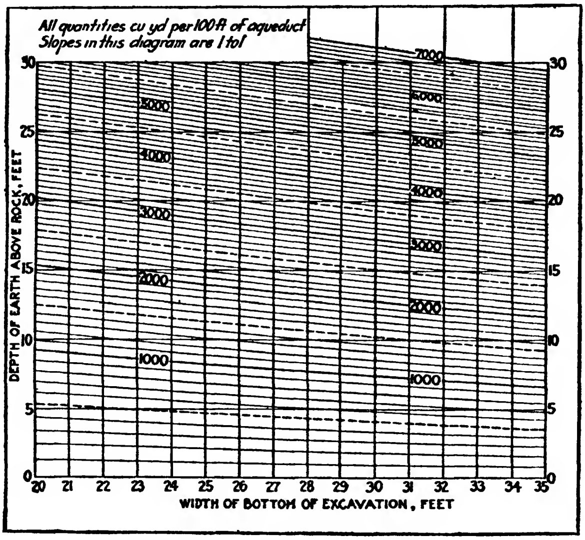


FIG. 149.—Earth quantities.
(Use with Fig. 148)

a number of years, when a slight leak developed, the cause of which had not been discovered at time of writing this.* The lead weighed 5 lbs. per sq. ft.; came in sheets 16 ft. long and 9 ft. wide. Sheets were burned together without solder. To insure the permanence of the lead and make satisfactory work, the bottom and sides of the aqueduct, against which lead was to be placed, were smoothly plastered, with $\frac{1}{2}$ in. of Portland cement mortar and then coated with asphalt. After the sheets of lead had been put in place, and burned together, they were covered with a thick coat of asphalt; then an 8-in. brick lining was put on. To protect the top of the bridge from weather, to prevent rain from soaking into the masonry, and to make a suitable finish, it was covered with granolithic, laid on 2 layers of roofing felt coated with coal tar.

Wellesley Arches and Echo Bridge of the Sudbury aqueduct were lined similarly to the Assabet bridge, but not for full hight, as it was not feasible in

*Longitudinal hair cracks appeared in brick lining. A wash coat of grout has been put on. Effectiveness not yet known. (Jan., 1916)

the upper part. The remedy has been effective; it was applied after many years of annoyance from leakage and freezing. Echo Bridge, carrying Sudbury aqueduct across Charles river, leaked badly at haunches after 35 yrs.; 9 semi-circular arches, 22 ft. 4 in. radius, carry brick aqueduct of horseshoe section, 9 × 7.5 ft. Cracks were found about 2 ft. either side of center line on invert, and directly over joints in roof stones of drainage gallery beneath. To remedy leaks, wetted perimeter was lined with sheet lead, $\frac{1}{16}$ to $\frac{1}{8}$ in. thick, weighing 3.5 lbs. per sq. ft. Lead was procured in 14-ft. squares and connected by oxyhydrogen flame to form continuous lining. On invert, $1\frac{1}{2}$ in. floor was laid on lead, composed of 1 Portland cement to 2 parts pea gravel, reinforced with No. 26 expanded metal, finished with granolithic surface of 1 : 1 mortar $\frac{1}{16}$ in. thick. Floor was vented by holes spaced 4 ft. apart, to prevent floating. No further leakage trouble. (E. R., Mar. 25, 1911.)

AQUEDUCT TUNNELS

Shapes. Most contractors prefer to excavate a tunnel to rectangular shape; but horseshoe type is better adapted to withstand external heads. When driving a tunnel through rock of horizontal stratification, rectangular shape would leave a slab of rock self-supporting until lined, while horseshoe type would cantilever out horizontal layers in arched roof, giving more safety. For tunnels under hydraulic pressure, circular cross sections are usually necessary.

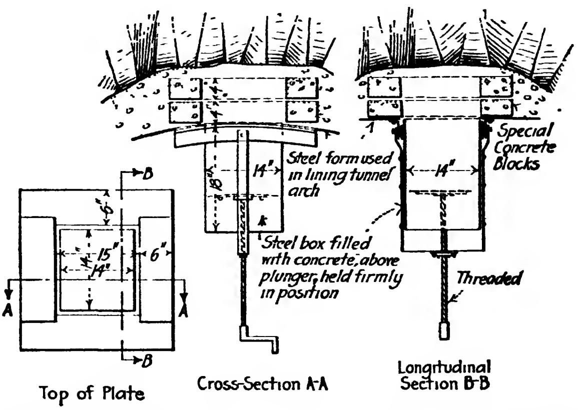


FIG. 150.—Catskill aqueduct. One method of placing last concrete in key of tunnel arch at a closure.

Relation of Depth of Cover to Hydrostatic Pressure Head. New Croton aqueduct, ratio of depth of cover (rock and earth) to pressure head varies from 0.572 to 0.703. On Catskill aqueduct, New York, minimum allowed rock cover is 150 ft.; maximum unbalanced bursting pressure, 420 ft.; maximum external hydrostatic pressure, tunnel empty, 1100 ft. (Hudson river crossing); maximum depth below hydraulic gradient, 1500 ft. All these figures refer to pressure tunnels, so-called, below hydraulic gradient. (See Table 71.)

Shafts.* *Shape of Shaft in Rock.* Circular shafts are no more difficult or

* For further information see "Practical Shaft Sinking," by Francis Donaldson (McGraw-Hill), 1910.

expensive to sink than rectangular of same size. Long rectangular shafts are easier and cheaper to excavate, due to larger quantity which can be taken out as bench, except that corners are expensive and troublesome. Circular shafts are free from corner difficulty and, yard for yard, often cost no more than rectangular. Elliptical shafts are better than either, having advantages of both; are of good section to sink through soft, water-bearing ground as they resist hydrostatic pressure well. Circular shafts are best for waterways, as downtake, uptake and drainage shafts of pressure tunnels; circular shafts were used extensively on Catskill waterworks of New York City.

Shaft and Tunnel Excavation. Methods have changed radically in last few years due to introduction of pneumatic hand drills. Formerly, when trimming had to be done by hand and was expensive, it was good practice to set heading holes well outside neat line to do away with trimming. With pneumatic hand drills trimming is greatly cheapened; it is now good practice to set main holes closer and do a great deal more trimming, resulting in much more accurate excavation (diminished breakage), with correspondingly reduced quantities of masonry.

Grade Tunnels: Ground-water Admission. Catskill aqueduct grade tunnel linings are not designed to resist external water pressure; excepting at a few places where quality is objectionable, ground water is admitted freely by venting under-drains and by building weepers into side walls. Design is based on assumption that ground-water head will exceed head of water in aqueduct except at portals, where tunnels are made tight and stronger.

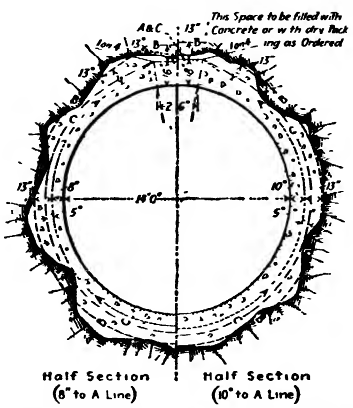


FIG. 151.—Concrete-lined pressure tunnel, Catskill aqueduct.
(Diameters 11′ 0″ to 16′ 7″)

Pressure Tunnels: *Brick Lining.* Tunnels of Niagara Falls Power Co. were lined with vitrified brick, after due consideration of concrete; four courses on American side, inner course being of especially hard vitrified brick; on Canadian side, one layer of vitrified brick backed with concrete; total thickness, 20 in. Latter is not approved as there is no opportunity for proper bonding. No erosion at Niagara to date. John Bogart (T. A. S. C. E., Vol. 31, 1894, p. 317), discussing Niagara tunnels, says various experiments with sand-blast showed that hard-burned brick surpassed in resistance to abrasion any substance practicable for tunnel lining. Brick lining of Cincinnati waterworks land tunnel consisted of 8.5 in. (2 rings) of special vitrified shale brick in 1 : 2 Portland cement mortar, backed with 1 : 2 : 4 Portland cement concrete, packed solid against wall all round, and grouted in wet ground. Special tunnel brick was voussoir shaped, and had 2 longitudinal grooves in each bed. Resident engineer, J. A. Hiller, considered parallel-faced brick without grooves quite as good for tunnels over 7 ft. diam.; these special features add slightly to cost. Chief engineer Benzenberg, on other hand, thought it advisable to keep down joint thickness by voussoir shape,

and considered grooves added to tightness of joint. Cement for grouting amounted to about ⅛ barrel per lin. ft. of wet ground; neat cement used.

Concrete lining, Catskill aqueduct, has various thicknesses, see Table 71, corresponding to various diameters and heads. These thicknesses represent the distance from waterway to "C" line. Rock is allowed to project 5 in. within this so-called "C" line (Fig. 151), but no timbering left in place can

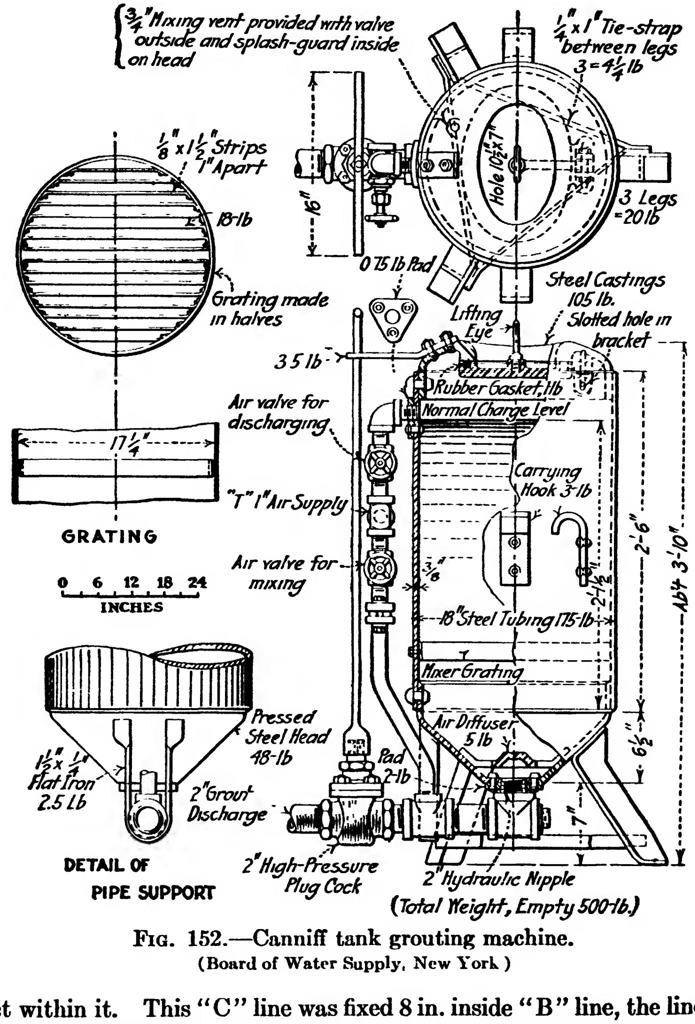


FIG. 152.—Canniff tank grouting machine.
(Board of Water Supply, New York.)

project within it. This "C" line was fixed 8 in. inside "B" line, the line about which the excavation is averaged. This "B" line was fixed from a study of breakage in existing tunnels as an average payment line. Contractor is not paid for excavation beyond this line; all excavation and concrete are measured within this line. Concreting was done in 5 stages: (1) Placing invert, about 5 ft. wide, upon which rails were laid for running the forms for remainder of circumference. (2) Placing walls of tunnel up to or slightly above center line. (3) Placing arch, excepting a 2.5-ft. key at crown, where concreting was most

difficult. (4) Concreting key from end of form, or making closure by special device, Fig. 147, or other means. (5) Grouting the dry packing, if any, and all seams and voids behind the lining, first at low pressures and finally at high pressures.

Grouting a Wet Tunnel. A portion of the Rondout pressure tunnel, Catskill aqueduct, was driven on an up-grade of 15 per cent. through badly folded and crushed strata of High Falls shale and Binnewater sandstone. These porous strata contained much water, which drained into the tunnel; inflow in 350 ft. of tunnel was 1950 gals. per min. (42 mgd. per mile) when first driven (Dec. 1, 1910) diminishing to 1000 gals. per min. (21 mgd. per mile) at time of grouting (Jan. 27, 1912). Water oozed and spurted in over entire area. The resulting cascade down the incline gave the impression of a good-sized trout stream. It was finally decided to place a 24-in. concrete lining through the wettest portion (175 ft.) inside of a steel shell; the space between the shell and the rock was dry packed, thus acting as an immense blind drain. Water collected in the dry packing was drained through 6-in. branch pipes into an 8-in. steel pipe buried under the invert, and discharging through a valve at the bottom of the wet stretch. This allowed the side walls and arch to be placed in perfect dryness, provided for the collection and disposal of water, and allowed maximum elasticity in grouting. The necessary steps in this work were: (1) laying the 8-in. steel pipe and Y-branches; (2) laying the invert; (3) erecting and dry packing the steel shell; (4) placing lining; (5) low-pressure grouting; (6) high-pressure grouting. The average thickness of the invert was somewhat over 3 ft. The wooden invert forms were calked with oakum, cement and plaster of Paris, thus forming a cofferdam 175 ft. long and about 6½ ft. wide. The water from the sides and roof ran along outside the forms and was diverted at intervals into the 8-in. pipe by small dams. After the possibilities of this work had been exhausted, there was still an excessive amount of water flowing with great velocity down the invert, due partly to springs in the bottom, partly to leakage through seamy rock under the forms, and partly to the difficulty of making tight the irregular contact between the forms and rock. Accordingly, the invert was divided into several stretches of 25 to 40 ft., by tight wooden bulkheads, and each stretch treated separately. A sand-bag dam was formed about 2 ft. upstream from the bulkhead and this water turned into the pipe through a hole cut through its top. Leakage through the dam was bailed out of the space between it and the bulkhead. The bottom was "dried up" wherever possible by "panning," but in most cases it was necessary to lay broken stone drains, covered with cement bags. At least 2 grout pipes were provided for each pan and drain to insure circulation of grout. The pan is a sheet of metal suitably bent to shape, laid over a spout of water. This work was tedious, but once com-

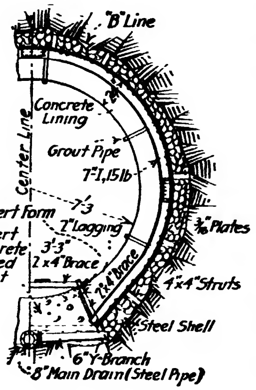


FIG. 153.—Rondout pressure tunnel. Lining a wet section.

pleted, the concreting was done without difficulty. Stretches of the tunnel north and south of the shell were grouted so as to drive all the water from the porous strata into the shell. Had the shell been grouted first, water might have been driven to stretches of tunnel north and south of it. This made it necessary to build very tight bulkheads or grout cut-offs at each end of the shell. Grout pipes within the shell were placed in two distinct systems: (1) low pressure, to fill dry packing; (2) high pressure, to shut off the water. Pipes for high pressure grouting were in seams where there were or had been large flows. Holes were drilled into seams 3 or 4 ft. deep and pipes set in the holes. Where a pipe entered rock, a mortar seal was formed so that grout from the dry packing could not enter the pipe unless it traveled back through the rock. Before grouting the dry packing, the valve in the 8-in. drain was closed and valves were placed on all the grout pipes into the dry packing, and closed. The packing filled quickly with water. All deep-seated pipes and a few pipes opening into the packing at top of the shell were left open to prevent accumulation of hydrostatic pressure. This formed a pond of still water into which grout was discharged without washing. Grouting the dry packing was begun at the lower end of the shell. Four Canniff grout machines were mounted on one platform; grouting was carried on simultaneously on both sides the shell. As the level of the grout rose it could be followed by opening grout pipes. The deep-seated pipes in general showed grout as soon as reached by the grout in the dry packing, and were at once closed; 7697 batches of neat cement grout (1.2 cu. ft. per batch) were forced into the packing in 10 shifts, the pressure being finally raised to 300 lbs. per sq. in. at top of shell. Maximum quantity for one 8-hr. shift was 1207 batches. The deep-seated pipes, which were closed when they first showed grout, were opened after the grouting of the packing had advanced a short distance beyond. In general, they ran as much, or more, clear water as before they were closed, thus preventing hydrostatic head on the soft grout. After the grout in the packing had set a few days, grouting the deep-seated pipes was begun at the bottom of the shell. A very thin grout was used ($\frac{1}{4}$ bag cement to 106 quarts of water). Holes took from a few to a maximum of 533 batches under a pressure of 300 lbs. and were closed by a batch or two of thick grout. The leakage on the incline, which had been 1000 gals. per min., was reduced to 8. The level of the water in the rock reservoirs as measured by a pressure gage in the tunnel, increased slowly, after leakage into the tunnel was stopped, from about 58 ft. above the shell when the grouting was completed, to 106 ft. 10 days later.

Grouting Wachusett Aqueduct Tunnel. A 2-mile stretch of Wachusett tunnel (1897), which might be under pressure, was dry-packed over the arch, where concrete work is difficult and expensive. Cut-off walls were built at 50-ft. intervals across this dry packing, so that it would not act as a drain to underground water. After the arch concrete had set, the dry packing was grouted through grout pipes left through the lining. A small hand force-pump gave 40 to 50 lbs. pressure. No vent pipes were used, after their uselessness had been demonstrated. It was possible to judge completeness of grouting by sweating on the interior surface. Grouting is essential to secure full and unyielding bearing for the lining at all points. Horace Ropes, engineer in charge, preferred grouted dry-packing to a solid concrete arch for: (1) The

Table 70. Pressure Tunnels for Water

(Catskill Aqueduct pressure tunnels on p 289)

Name	Location	Size, ft †	Hydrostatic head, ft.	Minimum depth below ground surface, ft	Lining: Kind	Lining: Thickness	Grouting	Geologic formations	Length, ft.	Shafts: No.	Shafts: Depth, ft.	Remarks
First Chicago*	Chicago, Lake Mich	5	70	35(e)	Brick	2 rings 4" brick	No	Stiff blue clay	10,567	2	70 75	Gas explosions
First Cleveland*	Cleveland, Lake Erie	5	90	54(e)	Brick	9"	No	Blue clay, quicksand	6,662	2	67 90	Gas and quicksand caused 20° deflection horizontally
Second and third Chicago*	Chicago, Lake Mich	7	70	35(e)	Brick	2 rings 4" brick	No	Stiff blue clay	10,460 10,908	2	75	These tunnels enter same crib as first tunnel
Mississippi River	Falls of St Anthony	35 × 4	— —	— — —	Concrete	— — — — —	— —	Soft sandstone	2,300	1	75	35' high; tunnel filled with masonry to stop underground stream
Dorchester Bay.	Boston, Mass.	7.5	157	118	Brick, concrete backing	12" brick	No	Clay, slate, conglomerate	7,000	3	157(b)	Water course, 2 mi wide. Discharge for sewage
68th St Intake*	Chicago, Lake Mich	5 6 7	— —	14(e)	Brick	12"	No	Clay	5,026 12,153 8,132	3	60(b)	Many deflections in alinement
New Croton Aq	New York	12.25 10.25	437	39	Brick, concrete and rubble backing	3 or 4 rings brick	Yes	Gneiss, some limestone	36,400 ±	15	426(a) 139(b) 80(c)	Water course, 400' wide
Gould's Swamp Siphon	New Croton Aq, N Y	14.25	70	105	Brick, concrete and rubble backing	3 rings brick	Yes	Gneiss	1,169	3	119(a) 78(b) 54(c)	Part of downtake shaft on incline
Washington Aq	D C	8.21 × 9.88	174	46	Brick, rubble backing	3 rings brick	Yes	Schists	20,696	8	170(a) 123(b) 60(c)	500' cast-iron lining, 9' diam. No work between 1888 and 1895
Four-mile Intake*	Chicago, Lake Mich	8 6	77	40(e)	Brick	13"	No	Clay and quicksand	9,139 25,200	3	75(b)	Quicksand caused substitution of two 6' tunnels for 8' tunnels inshore; united into 8' tunnel
Vyrnwy (Liverpool Supply)	Mersey Riv, England	10	359	29(e)	Cast iron	$1\frac{1}{4}$" plate, 6" flange	No	Gravel, sand and clay	810	2	53(b)	Water course, 805' wide. Shield used. Water pipes placed in passage
Second Cleveland	Lake Erie	7	— —	— — —	Brick	3 rings	— —	Clay, quicksand	9,177	2	115 145	Gas and quicksand interfered. Detours made
Milwaukee Intake*	Lake Mich	7.5	115	97	Brick	4 rings	No	Very hard clay, gravel, boulders, quicksand	3,200	3	80(b)	Water 1640' from shore necessitated deflecting tunnel sideways and upward

* Not strictly pressure tunnels, being intake tunnels pumped from shaft at one end
(a) Max depth (b) Average depth (c) Min depth (e) Measured from invert
† If circular, the one figure gives diameter of waterway, if not circular, extreme horizontal and vertical dimensions of waterway are given.

Table 70. Pressure Tunnels for Water.—*Continued*

Name	Location	Size, ft †	Hydrostatic head, ft.	Minimum depth below ground surface, ft	Lining		Grouting	Geologic formations	Length, ft	Shafts		Remarks
					Kind	Thickness				No.	Depth, ft.	
Lake View Intake*	Chicago, Lake Mich	6	70	45	Brick	8″	No	Clay, rock	10,000	3	80(*b*)	Many deflections in alinement
Northeast Intake*	Chicago, Lake Mich	10 (external)	99	64(*c*)	Brick	18″	No	Clay, sand	14,025	3	99(*b*)	
Third Cleveland*	Cleveland, Lake Erie	9	109	60(*c*)	Brick, masonry backing	13″ brick	No	Hard clay, gravel and sand, quicksand	26,040	4	122(*a*) 118(*b*) 112(*c*)	Several gas explosions; workmen killed
Cincinnati Intake*	Cincinnati, Ohio Riv.	7	146	52	Vitrified brick	15″	— —	Limestone, shale	1,426	2	149 143	Measurements refer to high water. 70′ range from low to high water
Waterworks Land Tunnel	Cincinnati, Ohio	7	160	80	Brick, concrete backing	2 rings brick, 8½″	Yes	Limestone, interbedded with clay and shale	22,250	5	160(*a*) 132(*b*) 110(*c*)	Tunnel is from 300′ to 1000′ from Ohio River. See p. 288
Champion Mill*	Plainsdale, Mich.	5.5 × 6.5	55	30	None	. . .	No	Sandstone and conglomerate	1,015	1	45	Under Lake Superior. 3 fissures in rock Drill holes pierce roof for water supply
Adventure Mill*	Lake Superior	7 × 5	56	35	None		No	Sandstone	887	1	93	Drill holes pierce roof for water supply
Kaw River.	Kansas City, Kan.	7	132	72	Brick, concrete backing	9″ brick	No	Hard soapstone	1,125	2	161 113	
Shawinigan Falls.	Canada	13	— —	— — —	Reinforced concrete	15″	— —	Blue clay	1,000	1	— — — —	Conducts water to turbines through 50′ ridge
Torresdale.	Philadelphia	10.5	120	85	Brick, concrete backing	2 to 5 rings brick	Yes	Gneiss	13,809	11	136(*a*) 112(*b*) 110(*c*)	
Los Angeles .	California	5	80	116	None	. . .	No	Solid rock	2,500	1	116	Infiltration gallery. Wells pierce roof
New Southwest*	Chicago, Lake Mich	13.67 × 14	130	98	Concrete	12″	No	Limestone, clay and quicksand	12,180	3	150 103 98	
Lake Erie	Buffalo, N Y	14.75 × 15	— —	— — —	Concrete	18″ floor 24″ roof	Yes	Seamy rock	6,000	2	75	

support for the masonry is more perfect as it is possible to force grout to the top of all cavities which could not be concreted; (2) it is more water-tight; concrete at top of arch could not be as liquid as elsewhere, and ramming is awkward; (3) it is rapid and cheap.

Method of Constructing Metropolitan Waterworks Tunnel under Chelsea Creek, Boston (1900). Driven by open-shield, 9 ft. in diam., composed of two circular ribs of angle irons and plates, connected by plates and braced by two I-beams, one vertical and the other horizontal. This frame was covered by plates $\frac{3}{4}$ in. thick, 4 ft. long, projecting over frame, but not sharpened; shield was used only for supporting the earth as excavation proceeded. It was moved by 6 hydraulic jacks arranged to be worked simultaneously or individually, as required to keep proper alinement. After the shield had advanced 15 to 18 in., jacks were removed and wood lagging put in, of 2-in. stock, 8 segments to the circle, and 4 in. thick. Lagging segments were spiked to, and broke joint with, the ring previously set. Exposed surfaces of excavation and the wood lining were promptly smeared with clay, to prevent waste of air. Wood lining took the thrust of the jacks and supported the walls until about 8 ft. of tunnel were ready for brick lining. Before bricklaying began, the clay was scraped off the wood lining and the whole given a wash of Portland cement. The brick ring was built solidly against the wood lining all around and was 12 to 16 in. thick, according as the wood lining varied from true alinement.—(C. M. Saville, engineer in charge.)

Leakage. *Cincinnati waterworks land tunnel* had a leakage inward of 10 gals. per min. for 22,250 ft., or 3400 gals. per day per mi.; later gagings showed 5800 gals. per day per mi., an increase of 70 per cent. in 8 months between the first and last gagings. The leakage concentrated in 1100 ft. of tunnel was at maximum rate of 117,500 gals. per day per mi. Leakage outward varied from 10 to 42 gals. per min.; maximum, 14,000 gals. per mi. per day, for the entire tunnel, or 290,000 gals. per day per mi. of the wet stretch. Maximum outward leakage occurred under a head of 34 ft.; inward, under an estimated head of 75 ft. The outward leakage of 10 gals. per min. was obtained under a head of 23 ft. for the first test, and 33 ft. for the fourteenth test, 1 month later. The reasons for the tightness of this tunnel are: (1) tightness of the rock, but 5 per cent. of which showed leakage before lining; (2) non-absorbent and doubtless impermeable character of the brick used; (3) the slow-setting mortar facilitated the complete filling of the joints; (4) exceedingly rigid inspection; (5) the large percentage of limestone dust probably made the concrete dense; (6) careful protection of the masonry from water during setting; (7) thorough grouting in wet ground.

In the *East Boston Subway tunnel*, a $1\frac{1}{2}$-mi. stretch leaked 25 gals. per min. (24,000 gals. per day per mi.) before grouting; 7 gals. per min., or 6700 gals. per mi. per day after grouting.

Jersey City Conduit, according to J. W. Hill (Proc. Engrs. Club of Philadelphia, 1905) showed a leakage during construction of 134,000 gals. per day, or 154,000 gals. per day per mi., for Section 1, of which 1600 ft. was a tunnel through shale and sandstone, and 3000 ft. in open cut. The conduit is concrete-lined, with an internal diameter of 8 ft. 6 in.

Table 71. Pressure Tunnels of Catskill Aqueduct

Name	Location	Diam.	Head, ft.		Min depth, ft., below surface	Lining thickness†	Geology	Length, ft (horizontal)	Shafts			
			Total on center line	Unbalanced on center line*					No.	Depth, ft. Max	Av.	Min.
Rondout.. .	Catskill Aq. north of N. Y.	14′ 6″	726	239	345	15″ and 17″	Shales, limestone, sandstone, quartzite	23,608	8	712	520	374
Wallkill..	"	14′ 6″	544	279	265	15″ and 17″	Hudson River shales and sandstone	23,391	6	483	387	350
Moodna.	"	14′ 2″	630	390	270	13″ and 15″	Shale and granite	25,200	7	586	451	342
Hudson..	"	14′ 0″	1,517	417	1,100	13″ and 15″	Granite	3,022	2	1,185	1,172	1,159
Breakneck	"	14′ 0″	600	405	195	17″ and 19″	Granite	783	1		625	
Croton .	"	14′ 0″	515	170	345	13″ and 15″	Granite	2,639	2	542	523	505
Yonkers	"	16′ 7″	155	55	100	15″ and 17″	Gneiss	12,978	4	155	130	106
City Tunnel.	Catskill Aq. in New York.	15′ 0″	633	295	210	13″ and 15″	Gneiss and schist	40,760	11	474	308	216
		14′ 0″	653	262	208	13″ and 15″	Gneiss and schist	23,890	6	446	274	215
		13′ 0″	459	253	200	13″ and 15″	Gneiss and schist	6,840	2	223	218	213
		12′ 0″	988	252	204	11″ and 19″	Fordham gneiss, Manhattan schist, Inwood limestone	10,550	2	741	730	708
		11′ 0″	1,005	295	308	11″ and 17″	Fordham gneiss, Manhattan schist, Inwood limestone	13,590	4	715	517	318

* Ground water assumed at surface. † See page 283.

The *New Croton Aqueduct* tunnel had a leakage outward in 7 mi. under 130 ft. head of 225,000 gals. per day, or 32,100 gals. per day per mi. The leakage inward in 25 mi. of tunnel lined with brick (and concrete rubble in places) was 4 mgd. or 160,000 gals. per day per mi.

Catskill aqueduct pressure tunnels in rock are being shown by preliminary tests to be satisfactorily tight. Results of final tests are not yet available.

Tunnels: Capacity, Lined vs. Unlined. With two rock tunnels of large size and same slope, one unlined would require twice the cross-sectional area of one lined with smooth masonry to carry same quantity of water.

Unlined portions of Wachusett rock (granite and diorite) tunnel have given little trouble since aqueduct has been in service (1898). No large masses have fallen, but small quantities of chips are found on invert each time aqueduct is cleaned. Loss of head due to roughness of rock is great.

AQUEDUCT MAINTENANCE

Fouling. Compensation must be made for the greater fouling of aqueducts near the head-works. Observations show that a point exists, downstream from which fouling is practically constant. The fouling of Sudbury aqueduct extends downstream about 1000 ft., decreasing rapidly at first, and then more slowly; beyond 2 mi. from the head-works, fouling is constant. Fouling reaches its ultimate effect on flow in 6 or 7 months. The decrease in capacity cannot be charged to diminution in cross-section alone; the fouling layer is principally Polyzoa, which intercept and hold diatoms and other organisms together with much amorphous matter. When the aqueduct is drained this appears as dark brown slime, usually not over $\frac{1}{16}$ in. thick; when submerged, the floating tentacles may occupy 5 times as much space; even so, the area of the aqueduct would not be diminished 1 per cent. Nevertheless, their removal after 1 yr. increases capacity 11 per cent. (max. 14 per cent.), as shown by current meter gagings before and after cleaning. In aqueducts carrying the same quantity, with the same degree of fouling (neglecting the small decrease in area and the effect of viscosity) the loss of capacity would vary directly with the wetted perimeter, and with some power of the velocity. The distance of the greater fouling, corresponding to the 2 mi. for Sudbury, would vary in other aqueducts in direct proportion to the volume of water passing, and in inverse proportion to the area for growths to fasten on.

Experience on Cochituate aqueduct, Boston (brick lining), is that deposits of slime and dirt allowed to accumulate give place to growths of spongilla, which after a year or two become hard, with long finger-like projections, forming serious obstructions to flow of water; maximum reduction of capacity was 12.5 per cent. For Wachusett aqueduct,* Boston (brick lining below springing line, concrete above), 10 per cent. reduction in flow was measured at end of the first year, with coat of slime about $\frac{1}{16}$ in. thick, but no spongilla. New Croton aqueduct,† New York (brick lining), after 9½ years' constant use without cleaning showed a reduction in flow of 14 per cent.

Freedom from Fouling of Concrete Lining. The deterioration in carrying capacity of concrete conduits is small, probably less than for brick. In 20

* See Fig. 144. † See Fig 142.

mi. of cement-lined pipe taken up in Northern New Jersey, no instance of growth or deposit was found, the pipe being apparently as clean and free from slime as on the day laid. An examination in 1911 of the Jersey City concrete aqueduct built in 1903 showed the same characteristic, the bottom and sides being free from slime.

Growths in Aqueducts and Canals. S. Fortier states that algæ cause losses in carrying capacity altogether disproportionate to the cross-sectional area occupied; algæ appear again shortly after being scraped off. The Genus Cladophora, a long-fibered variety, causes most trouble in irrigation canals; it is said to be related to Conferva Bombycinum, which requires about 1 part copper sulphate to 1 million parts of water for extermination. This type does not seem to grow in darkness. In the concrete-lined canal of Arizona Power Co., tunnel portions were clean, while the canal portions contained algæ growths, although velocity was the same in both. Growth might be discouraged by painting concrete lining dull black, for observations show more profuse growths on light surface than on dark. No ordinary stream velocity stops growths; algæ are not so numerous in silt-laden waters; water temperature seems to have little effect. (Trans. A. S. C. E., Vol. 71, 1911, p. 186.)

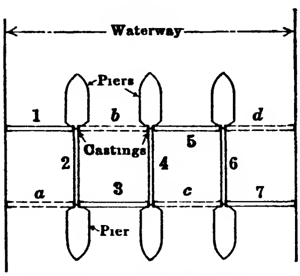
Cleaning Aqueducts. Wachusett and Weston aqueducts were formerly cleaned about once a year, and Sudbury twice, restoring the flowing capacity to within 2 or 3 per cent. of that when first put into service; the decrease in capacity becomes about 10 per cent. by end of year. The deterioration takes place gradually and nearly uniformly throughout the year, according to some observers, but according to others—and more probably—most of it occurs within 5 months after being returned to service. Of recent years, cleanings have been infrequent. New York aqueducts are not cleaned regularly; only parts of any one have ever been cleaned, because experience indicated little advantage could be secured, and the service could not be interrupted long enough.

Cleaning and Repairing Sudbury Aqueduct (Annual Report, Metropolitan Water and Sewerage Board, 1913): Two gangs, one beginning at each end of section to be cleaned worked toward the center. One gang used water and hand brooms in cleaning the sides and top of the aqueduct, a wagon with a tank body being used when cleaning the top. The second gang made use of a gasoline power sprayer, by which water under 200 lbs. pressure was discharged against the interior surface of the aqueduct. This method proved more efficient, more economical and quicker than the hand method. The cleaning of the bottom was done with push brooms operated by hand. The ironwork of the gates and stop-plank grooves in the screen and terminal chambers was thoroughly scraped, dried by the use of torches, and painted with two coats of red lead and oil. When the aqueduct was first emptied it was found that water was flowing in through small crevices in the brick and concrete, the greater number of which were where the aqueduct is generally in tunnel. In this section 1400 leaks in the tunnel lining were repaired by driving and calking wedge lead, tea lead or lead wool into the joints.

AQUEDUCT APPURTENANCES

Plug Drain Valves for Chambers. For drainage, gate and other chambers, 6-in. valves, with rubber plugs and rising stems have proved satisfactory; iron guides should not be used for the plug; they rust so badly that the disk cannot be lifted. A non-rising stem necessitates a hole through disk, and if the screw threads do not fit perfectly, leakage results.

Screens at Inlets. Screens are necessary where any surface supply is stored. A general arrangement at headworks is: Guard the first inlets by outside racks of metal or wood bars, set edgewise 4 to 6 in. apart, to prevent larger objects from entering; inside the chamber, place coarse copper screens, about 1-in. to $1\frac{1}{2}$-in. mesh, to remove leaves and larger fish; finally guard entrances to pipes, pumps, or filters by screens having 4 to 6 meshes per in., arranged to be frequently removed and cleaned. Screens should be in duplicate in each passageway, where complete guard is required, so that one set can be placed before the other is removed for cleaning.



Full Lines show Normal Position. Alternate Position, Screens in Bays $a, 2, b, 4, c, 6, d$

FIG. 154.—Re-entrant bay arrangement of screens.

Screens in aqueducts should be placed in chambers, giving access for removing accumulations before they become serious obstructions. Clean screens of ample area cause no loss of head. Clogged screens will be torn out if of fine wire.

Screen areas should be liberal. For screens guarding entrances to pipes, wire should be No. 14 or $\frac{1}{16}$ in. Design of screens and frames, and apparatus for lifting them (N. Y. Board of Water Supply, Fig. 155) has proved satisfactory.

Instead of wire staples, brass clips spaced 15 in. apart should be used to fasten the trash baskets to screen frames so that they can be removed easily.

Revolving screen, Roosevelt Canal,* was installed to remove floating débris at penstocks. Screen is in form of truncated cone, smaller end (7.5 ft. diam.) down-stream; axis, (16.5 ft. long) inclined so that most of smaller end is above high-water line and over 50 per cent. of larger end (11 ft. diam.) is submerged. No. 18 galvanized wire, $\frac{1}{2}$-in. mesh, is supported by $\frac{5}{8}$-in. and $\frac{3}{4}$-in. circumferential rods, in turn supported by sixteen 4-in. $\times$ $\frac{3}{4}$-in. bars. Water enters at larger end which is free of interior bracing. Screen is held to position by an exterior trunnion ring, 5 ft. $\pm$ from larger end resting on trunnion either side of center line, and by a shaft fastened to smaller end. This shaft passes far enough into screen to support one end of collecting trough; other end slopes toward, and is supported by pier upstream from screen. Screen is turned $\frac{1}{60}$ r.p.m. through gearing and ratchet ring at upper end by 3.hp. motor. Area of submerged screen is 232 sq. ft., when canal is discharging 250 cfs. Revolutions carry débris to top of screen whence it drops into this trough. Periodical discharges from tank above screen wash out trough and free screen of any adhering débris.†

* F. Teichman, T A S C E, Vol 60, 1908

† Later advice shows that revolving screens may be not as economical as rackbar. Tank had insufficient capacity and elevation; men have to use rakes; grass and moss cling to screen.

Even with double screens, close together, with reasonable care in cleaning, some fish and trash will get into the pipes. The re-entrant bay arrangement of screens would not be suitable in many places, since the large spaces between screens put across the bays during cleaning and the regular screens, would contain fish and refuse that could not be conveniently removed. Inclined screens are objectionable since (1) they cannot be used in deep water; (2) a large proportion of the fine dirt is rubbed through during cleaning, which is done by brushing. The Metropolitan Water Board standard screen is of copper (No. 16, Birmingham wire gage) of 6 meshes per in. Frames are of white pine, fastened with tree nails. The netting is stapled to the upstream side of the frame, and a ½-in. board, full width of frame, is placed over the staples. A most important part of the equipment is the "dirt catcher"—a wire basket of the same netting as the screen, fastened to the bottom of the screen. Baskets were sometimes made detachable on Catskill aqueduct. The G. E. Winslow Co., gage makers, devised for some New Jersey water-

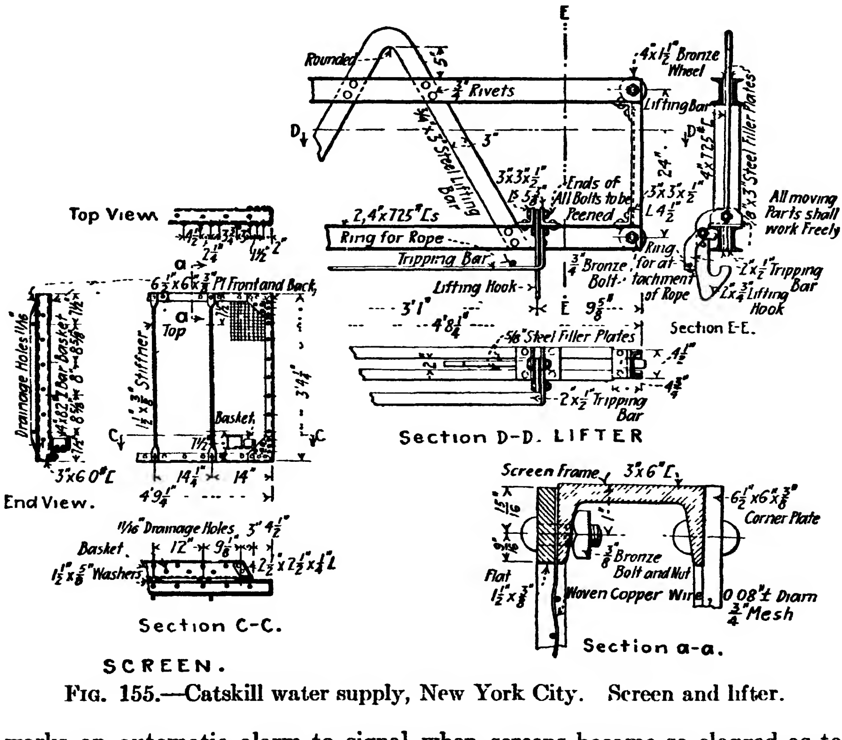


FIG. 155.—Catskill water supply, New York City. Screen and lifter.

works an automatic alarm to signal when screens became so clogged as to cause a loss of head which threatened to break them. A concrete-lined channel, 24 in. deep, at the bottom of screens serves as a sand trap.

Stop Planks. Grooves for stop planks must be smooth and straight. Instead of using single stop planks, fasten two or more together so as to form a shutter, which cannot come out of groove by tilting, while being raised or lowered. For great depths or large openings steel plate shutters with wooden bearing strips on edges are preferable to wood. Wooden stop planks

should be weighted so as to just sink. They may be given a few coats of boiled linseed oil or creosoted. Long-leaf yellow pine is a very suitable wood. Stop-planks,* Hinckley dam, N. Y. State Barge Canal, are built up of white-oak sticks, fastened between steel I-beams, in sections of convenient size. Each section is provided with bronze rollers to reduce sliding friction in both directions.

Floor covers over stop-plank and similar openings, if of wood, are easier to handle than if of metal, are fairly durable, and easy to replace, but if not frequently inspected, may become unsuspectedly weak, and so dangerous, from rotting on underside, while top remains sound. Rolled steel or thin cast-iron plates are preferable. Reinforced concrete slabs may be used where they do not have to be moved often; they are heavy. Wrought-iron or steel plates should be heavily galvanized or kept well painted. particularly on under side.

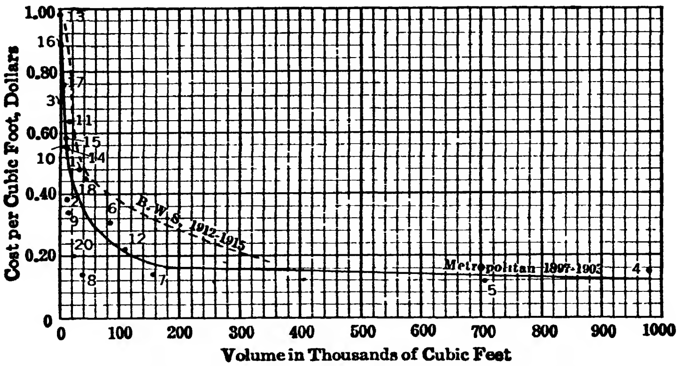


FIG. 156.—Cost of superstructures for water works structures

Cost of Buildings. The following 20 structures were built by Metropolitan Water Works, Boston, (10-hour day):

1	Dam gate house	Granite, brick lining
2	Aqueduct terminal chamber	Granite, brick lining
3.	Aqueduct gaging chamber	Granite
4	Pumping station	Limestone, granite trimmings, brick lining.
5	Pumping station	Sandstone, granite trimmings, brick lining.
6	Gate house	Limestone, brick lining
7	Buildings at pipe yard	Brick.
8.	Sewage pumping station	Sandstone, granite trimmings, brick lining
9	Gate house	Granite, brownstone trimmings, brick lining.
10	Gate house	Limestone, granite trimmings, brick lining.
11	Gate house	Granite, brick lining.
12	Water tower	Granite and concrete.
13.	Connection chamber	Granite, brick lining.
14	Siphon chambers	Granite, brownstone trimmings.
15	Siphon chambers	Granite, brownstone trimmings
16	Gaging chamber	Granite, brownstone trimmings.
17	Gaging chamber	Granite, brownstone trimmings.
18	Aqueduct terminal chamber	Granite, brownstone and limestone trimmings, brick lining.
19	Head and meter chamber	Granite, brownstone and limestone trimmings, brick lining
20.	Filter pumping station	Granite, brownstone and limestone trimmings, brick lining.

Diagram of Board of Water Supply costs (B W S) is based on the average bid for concrete-stone (cast stone, or synthetic stone), brick and stone superstructures, (8-hour day).

Gate-houses: Some Architectural Details. Red Conossera Spanish and similar tile roofs for gate-houses are satisfactory in appearance; but for weather-tightness in climate with high winds and drifting snow, they are not adapted, unless laid on steep slopes; even then it is necessary to have tight

*E N Feb, 15, 1912, p 282.

roof beneath. Terra-cotta tiles are broken by stones thrown by mischievous persons, by bullets, and sometimes by frost. Board of Water Supply, New York, has adopted strong reinforced concrete tiles to meet this trouble and for other reasons. (Manufactured by Am. Cement Tile Mfg. Co., Lincoln, N. J.) These tiles are about 24 × 32 in., 1⅛ in. thick. Roof frames are of steel protected from corrosion by heavy mortar coat. In some waterworks structures wood in roofs and other parts is rapidly decayed by dampness. Ample ventilation is a help. Windows should be of heavy wire-glass, with Monel metal or other rustless wire,* and in some situations must be protected by shutters, grilles, or strong wire netting. For very permanent construction set glass in metal sash or directly in the masonry. Strong rustless wire screens of coarse mesh are useful in chimney tops to keep out birds and other obstructions; also outside of windows to keep out birds and prevent breaking of glass by stone throwing. Doors should have sills an inch or more above floor. In some buildings freezing of condensed moisture on inside of building interferes with operation of doors, windows and other moving parts.

Metal Ladder Rungs should be 8 in. from wall, so as to come under instep with large boots; 6 in. not enough.

Current Meter Gaging Chambers have been built on some large aqueducts and so equipped that a current meter could be used with precision to determine the average velocity in any practical depth of water flowing. Then, the areas corresponding to depths being known, the quantity would be known. A slot is formed in the aqueduct arch and walls carried up to the top of the embankment, where a level floor is built and covered with a shelter. The meter, without guide vanes, is attached to a long stiff handle. On a suitable diagram, the cross-section of the aqueduct is divided into a large number of approximately square equal areas, arranged in horizontal bands. The meter may be held at the points in the water corresponding to the centers of these areas, one after another, and the average velocity thus obtained, or it may be steadily moved at such a rate as to be equal times in each area, beginning at one end of the bottom or top band of small areas. For the latter scheme two methods have been used: A machine may travel on tracks laid on the floor, transversely to the axis of the aqueduct, moving the meter horizontally and vertically as required, by automatic devices. By the other method the handle of the meter, held in a clamp fixed at a point suitably high above the axis of the aqueduct, is oscillated about the fixed point and slipped through the clamp by hand, the motion being timed by the beating of a metronome. To direct the meter along the right path a small guide wheel clamped to the handle below the fixed point travels along a guide path made of wood or metal and fastened to a plane transverse to the axis of the aqueduct and a suitable hight above the highest water level.

Air Vents. At gate chambers, drops and other places connected with aqueducts, pipe lines and other conduits where water falls or descends rapidly in the atmosphere, air is entrained, often in unsuspectedly large quantities. This air if allowed to be carried into the conduit is sure to accumulate in some place where trouble will be caused. Therefore liberal vents should be provided as near as is practicable to the points of entry; they should be so protected that they will not be stopped up by snow or by the freezing of vapor issuing with the air; furthermore they must be so guarded that sticks and stones cannot be thrown into them by mischievous persons. Wherever practicable, air should be prevented from so entering.

* On Catskill aqueduct structures, such glass was furnished by Pennsylvania Wire Glass Co., Philadelphia.

CHAPTER XV

PLATE METAL PIPES

RIVETED JOINTS

Longitudinal Seams. With low heads or pressures it is only necessary to design longitudinal seams for tightness, and strength will be excessive. Make the pitch of circular and longitudinal seams the same, as the cost will be the same even if rivets are added to the longitudinal seam. The lap joint is easier to make, but above $\frac{5}{8}$-in. plate it is hard to get a good job. Butt joint is satisfactory and stronger than lap joint for thicker plates. One or two longitudinal seams are used, depending on the diam. and thickness of pipe.

Circular Seams. Make number of rivets multiple of 4 so that they will be at 90° points. Single riveting is generally strong enough. Either lap or butt joints are acceptable to shops, no matter what the thickness of plate, the lap joint being easier to make. The question of circular seams is an economic one, based on gain in hydraulic properties due to the smooth interior with the butt joint, against a rough section, with alternate large and small sections, with the lap joint. Butt joints add from 0.3 to 0.5 ct. per lb. to the cost of the pipe. (See page 358 for temperature stresses.)

Rivet Holes should be $\frac{1}{16}$ in. greater than the nominal diam. of the rivet; the driven rivet should fill the hole. Specifications commonly forbid drifting; some experienced pipe makers believe that, with good plate, drifting to the extent of $\frac{1}{16}$ in. or even $\frac{1}{8}$ in. is not objectionable. Catskill aqueduct specifications: "Rivet holes shall be spaced with precision. Holes in $\frac{1}{2}$-in. and thicker plates shall be drilled from solid, or punched to approved size and reamed. Punched holes shall be clean cut, without torn or ragged edges, and in punching only the best and sharpest dies shall be used. All burrs or splits caused by punching shall be removed by countersinking. Corresponding holes shall, without enlarging, coincide to within about $\frac{1}{32}$ in., and all plates in which corresponding holes do not so coincide shall be rejected, unless the engineer shall be of opinion that conditions of stress are such at the point in question as to make more extensive enlargement safe. Drift pins shall not be used in forcing holes, but any perceptible lack of coincidence in acceptable plates shall be corrected by a sharp reamer or drill." Plates should be punched *from* the contact side.

Rivets. There is little if any difference in driving between soft steel and iron rivets, but steel rivets are better; the softer rivet has the lower shearing strength. The size of rivet for theoretical efficiency varies with type of joint and thickness of plate, being usually for lap joints about twice the plate thickness from $\frac{5}{8}$ in. up to $1\frac{1}{8}$ in. diam. of rivet. Maximum economy results when crushing strength equals shear. On one large work, the use of one

size rivet (1 in.) throughout proved more economical as the increased thickness of cover plate more than offset the decreased thickness of the main plate, in the butt joints (pipes 9 to 11 ft. diam.). For thick plates ($\frac{3}{4}$ in. and greater) $1\frac{1}{8}$-in. rivets are preferred, since small rivets cool before being thoroughly driven. Practice is divided on hot and cold rivets. The pinching of a hot rivet in cooling adds to strength as well as to tightness. Cold rivets are sometimes used for small and light work, and at least one large shop uses them regularly for large work; has used them successfully for some of the heaviest pipe made. With cold-driven rivets there is no layer of slag between rivet and plate; less lateral contraction and so a closer fit in hole. The principal danger, if the pressure of the riveting machine is not applied at right-angles to the plate, is that the head may be partly sheared off, but such defects are easily discovered, especially during the hydrostatic test, when such heads come off. This should not be considered characteristic of cold driven rivets, but merely accidental. Some makers can give lower price if cold rivets are used. Driving cold requires 4 or 5 times as much power.

Table 72. Pressures for Driving Hot Rivets (R. D. Wood & Co.)

Rivet diam., in.	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$
Pressure, tons	20	25	33	45	60	75	100	125	150

Shop Riveting. Table 72 is based on the rivet passing through only 2 plates, the combined thickness of which does not exceed the rivet diam.

Another manufacturer uses a driving pressure of 63 tons for 1-in. rivet, irrespective of thickness of plate; this brings the plates together well; a higher pressure would tend to deform the plates.

Rivet heads may be safely flattened to a thickness equal to $\frac{1}{4}$ the diam. of the rivet. The splitting of rivet heads depends on length of rivets, not diam., and is due to contraction in cooling. Full rivet heads can be tightened to prevent leakage, by applying pressure to the cold rivet. On Catskill aqueduct rivet calking was not allowed if the rivet was loose, unless the leak was slight and easily closed, otherwise the rivet was cut out, and a new one headed up tight while hot. Countersunk rivets are difficult to make tight, even by the best means yet devised; a number will leak in the first instance. Calking of outside heads only can be done, and if this does not stop leakage the rivet must be cut out (requiring the emptying of the pipe). To countersink rivets on the inside (to improve hydraulic qualities) adds about $\frac{1}{4}$ ct. per lb. to the pipe cost

All riveting in longitudinal seams should be done by hydraulic, compressed air or steam machinery. The pressure is retained until the rivet head has been perfectly formed and the metal has lost its red color. Suitably applied pressure holds the plates in close contact until the rivet is driven. Rivets should be driven at such heat as to give the best results.

Pitch of Rivets. Single and double riveting are used for lap, and double

Table 73. Rivet Gages and Edge Distances for Steel Pipes
Chicago Bridge & Iron Works

Rivet diam., in	Center of rivet to edge of plate, in.	Spacing of rivet lines, in.
$\frac{1}{2}$	1	$1\frac{1}{2}$
$\frac{5}{8}$	$1\frac{1}{8}$	$1\frac{3}{4}$
$\frac{3}{4}$	$1\frac{1}{16}$	2
$\frac{7}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$

and triple riveting for butt joints. Double riveting is 16 to 20 per cent. stronger than single. *Joint efficiency* is the ratio of stress to strength, using unit stresses based on tests. With $\frac{1}{2}$, $\frac{7}{16}$ and $\frac{9}{16}$ plates on the Catskill aqueduct, an edge distance of $1\frac{5}{8}$ in. was used with 1-in. rivets. In single-riveted lap joints, the minimum pitch of rivets depends on the value of tearing of the plate. The maximum pitch is determined by requirements of calking. One manufacturer has the general rule that the pitch of rivets in butt joints should not exceed 8 times the thickness of the thinnest butt plate. The minimum longitudinal pitch in a single-riveted lap joint is generally limited by the clearance between rivet heads, usually taken as $\frac{1}{2}$ in.

Costs depend on many conditions, but all variations of $\frac{5}{8}$-in. to $\frac{3}{4}$-in. rivets, double or single riveting, would be covered by 10 ct. per ft. of 48-in. pipe. Pipes with tapered, or "stovepipe," joints cost a little more than alternate inside and outside courses, and butt-strap joints much more, especially if the inside rivet heads are countersunk. Tapered rings require lay-out plates and make good index punching* impossible; also require the plates to be curved to a cone. The price for steel pipe can be $\frac{1}{4}$ to $\frac{1}{2}$ ct. a pound less, if liberal time be allowed for shop work.

Lap-riveted Joints. Lap length is reckoned from edge to edge of bevel. Length of lap is determined by the shearing value of the rivet through the plate, and should not be less than $T \div S$ times the effective area of plate between rivets; T = strength in tension, S = strength in shear. Boiler makers commonly assume the lap as $3d$,† probably because of the wedge action of the rivet. A single-riveted joint may fail by deficient plate bearing, shearing rivets, tearing of plate, rivet shearing through plate, and rivet breaking through plate. The breaking of the rivet through the plate depends on the lap and pitch and is difficult to calculate; it need not be considered in most cases, as the pitch must be close to insure tight work, and only where large pitches are used need breaking be considered. Butt straps require double the riveting and have double the chance to leak both at rivets and edges, to say nothing of the increased work and materials but are necessary for strength in many cases. The single-riveted longitudinal joints are seldom used in water pipes above 4 ft. diam. Double-riveted lap and double-riveted butt joints have the same strength, but the lap joint is cheaper. A joint having a double-riveted butt strap on the outside and a triple-riveted butt strap on the inside can be designed to have efficiency of about 85 per cent.

* Using an index plate. † = diam of rivet

Table 74. Double-riveted Steel Hydraulic Pipe*
Byron Jackson Iron Works, Inc.

Diam., in.	Thickness: U. S. wire gage	Thickness: In., Common fractions	Thickness: In., Decimals	Head, ft., pipe will safely stand	Weight† per lin. ft., lbs.
(1)	(2)	(3)	(4)	(5)	(6)
3	18	1/20	0.05	810	2.3
4	18	1/20	0.05	607	3.0
4	16	1/16	0.06	760	3.8
5	18	1/20	0.05	485	3.8
5	16	1/16	0.06	605	4.5
5	14	5/64	0.08	757	5.8
6	18	1/20	0.05	405	4.3
6	16	1/16	0.06	505	5.3
6	14	5/64	0.08	630	6.5
7	18	1/20	0.05	346	4.8
7	16	1/16	0.06	433	6.0
7	14	5/64	0.08	540	7.5
8	16	1/16	0.06	378	7.0
8	14	5/64	0.08	472	8.8
8	12	7/64	0.11	660	12.0
9	16	1/16	0.06	336	7.5
9	14	5/64	0.08	420	9.3
9	12	7/64	0.11	587	12.8
10	16	1/16	0.06	307	8.3
10	14	5/64	0.08	378	10.3
10	12	7/64	0.11	530	14.3
10	11	1/8	0.13	607	16.3
10	10	9/64	0.14	680	18.3
11	16	1/16	0.06	275	9.0
11	14	5/64	0.08	344	11.0
11	12	7/64	0.11	480	15.3
11	11	1/8	0.13	553	17.5
11	10	9/64	0.14	617	19.5
12	16	1/16	0.06	252	10.0
12	14	5/64	0.08	316	12.3
12	12	7/64	0.11	442	17.0
12	11	1/8	0.13	506	19.5
12	10	9/64	0.14	577	21.8
13	16	1/16	0.06	233	10.5
13	14	5/64	0.08	291	13.0
13	12	7/64	0.11	407	18.0
13	11	1/8	0.13	467	20.5
13	10	9/64	0.14	522	23.0
14	16	1/16	0.06	216	11.3
14	14	5/64	0.08	271	14.0
14	12	7/64	0.11	378	19.5
14	11	1/8	0.13	433	22.3
14	10	9/64	0.14	485	25.0
15	16	1/16	0.06	202	11.8
15	14	5/64	0.08	252	14.8
15	12	7/64	0.11	352	20.5
15	11	1/8	0.13	405	23.3
15	10	9/64	0.14	453	26.0
16	16	1/16	0.06	190	13.0
16	14	5/64	0.08	237	16.0
16	12	7/64	0.11	332	22.3
16	11	1/8	0.13	379	24.5
16	10	9/64	0.14	425	28.5
18	16	1/16	0.06	168	14.8
18	14	5/64	0.08	210	18.5

Diam., in.	Thickness: U. S. wire gage	Thickness: In., Common fractions	Thickness: In., Decimals	Head, ft., pipe will safely stand	Weight per lin. ft.,† lbs.
(1)	(2)	(3)	(4)	(5)	(6)
18	12	7/64	0.11	295	25.3
18	11	1/8	0.13	337	29.0
18	10	9/64	0.14	378	32.5
18	8	11/64	0.17	460	40.0
20	16	1/16	0.06	151	16.0
20	14	5/64	0.08	189	19.8
20	12	7/64	0.11	265	27.5
20	11	1/8	0.13	304	31.5
20	10	9/64	0.14	340	35.0
20	8	11/64	0.17	415	45.5
22	16	1/16	0.06	138	17.8
22	14	5/64	0.08	172	22.0
22	12	7/64	0.11	240	30.5
22	11	1/8	0.13	276	34.5
22	10	9/64	0.14	309	39.0
22	8	11/64	0.17	376	50.0
24	14	5/64	0.08	158	23.8
24	12	7/64	0.11	220	32.0
24	11	1/8	0.13	253	37.5
24	10	9/64	0.14	283	42.0
24	8	11/64	0.17	346	50.0
24	6	13/64	0.20	405	59.0
26	14	5/64	0.08	145	25.5
26	12	7/64	0.11	203	35.5
26	11	1/8	0.13	233	39.5
26	10	9/64	0.14	261	44.3
26	8	11/64	0.17	319	54.0
26	6	13/64	0.20	373	64.0
28	14	5/64	0.08	135	27.3
28	12	7/64	0.11	188	38.0
28	11	1/8	0.13	216	42.3
28	10	9/64	0.14	242	47.5
28	8	11/64	0.17	295	58.0
28	6	13/64	0.20	346	69.0
30	12	7/64	0.11	176	39.5
30	11	1/8	0.13	202	45.0
30	10	9/64	0.14	226	50.5
30	8	11/64	0.17	276	61.8
30	6	13/64	0.20	323	73.0
30		1/4	0.25	404	90.0
36	11	1/8	0.13	168	54.0
36	10	9/64	0.14	189	60.5
36		3/16	0.19	252	81.0
36		1/4	0.25	337	109.0
36		5/16	0.31	420	135.0
40	10	9/64	0.14	170	67.5
40		3/16	0.19	226	90.0
40		1/4	0.25	303	120.0
40		5/16	0.31	378	150.0
40		3/8	0.38	455	180.0
42	10	9/64	0.14	162	71.0
42		3/16	0.19	216	94.5
42		1/4	0.25	289	126.0
42		5/16	0.31	360	158.0
42		3/8	0.38	435	190.0

* Unit stress = 15,000 lbs. per sq. in.; joint efficiency = 71 per cent.
† Including laps and rivet heads.

Table 75.—Properties of Riveted Lap Joints for Steel Pipes and Tanks

Chicago Bridge & Iron Works

Thickness of plate, in.	Number of rows	½-in rivets			⅝-in. rivets			¾-in. rivets			⅞-in rivets		
		Eff.	Pitch	Sec.	Eff.	Pitch	Sec.	Eff.	Pitch	Sec.	Eff.	Pitch	Sec.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
¼	1	393	150	098	490	188	122	500	225	125	....	....	.
	2	654	180	163	700	250	175	650	250	160	....	....	...
	3	739	239	185	..		.	. .	....	. ..	....	....	....
9/32	1	349	150	098	435	188	122	500	225	140	500	263	140
	2	627	167	176	684	239	192	685	281	193	....	....	. .
	3	714	220	200	733	281	206	..	....	. ..	....	....	.
5/16	1	314	150	098	392	188	122	471	225	147	500	263	156
	2	600	157	187	663	222	207	708	300	221	679	312	212
	3	692	204	216	746	296	233	....	. .	....	.. .	..	.
11/32	1	286	150	098	356	188	122	428	225	207	500	263	172
	2	571	150	196	640	209	220	688	280	237	709	344	244
	3	673	191	230	727	276	250	745	344	256	....	....	..
	4	732	234	252	781	343	268	. .	. .	...	....	....	...
⅜	1	262	150	098	327	188	123	393	225	147	458	263	172
	2	523	150	196	615	198	231	669	264	251	707	340	265
	3	654	180	245	710	259	266	752	353	282	....	....	.
	4	714	220	268	767	320	288	.	.	.	. ..	....	.
13/32	1	241	150	098	301	188	122	363	225	147	423	263	172
	2	482	150	196	603	188	245	651	251	264	689	322	280
	3	635	171	258	693	245	282	737	332	299	..	....	
	4	700	207	284	705	302	286	785	406	319	..	...	.
7/16	1	224	150	098	280	188	123	337	225	147	393	263	172
	2	449	150	196	561	188	245	634	239	277	671	303	294
	3	619	163	271	677	233	296	722	315	316	756	409	331
	4	683	197	299	738	285	323	776	390	340	.	....	....
15/32	1				262	188	123	314	225	147	366	263	172
	2				524	188	246	618	229	290	659	292	309
	3				663	222	311	708	300	332	742	389	348
	4		..		725	271	340	764	370	358	787	469	369

For explanation, see page 301.

Wrought-iron riveted pipes of large sizes are of questionable economy as compared with steel pipes and there are few mills which can furnish the plates. Plates can be obtained up to 1 in. thick in large sizes from Glasgow Iron Co., Pottstown, Pa.; Central Iron & Steel Co., Harrisburg, Pa.; Moorhead Bros., Sharpsburg, Pa.; and Sligo Iron & Steel Co., Connellsville, Pa. Principal defects are: (*a*) Scabs, due to failure of pieces of component metal to weld into the plate, leaving the appearance of a splinter about 1/16 in. deep; (*b*) If a small piece of metal falls on to a plate in going through the rolls, and is rolled into it, this defect causes rejection; (*c*) Surface cracks radiate from holes or cinders; (*d*) Holes, not serious unless they penetrate through the plate; (*e*) Seams, streaks lengthwise of the plates, starting at holes which were rolled out, having their sides squeezed together; (*f*) Blisters, caused by cinders or other foreign substances preventing welding between two layers of the plate; not serious if small, or if they can be hammered flat; steel scrap in the welding pile causes blisters; (*g*) Cinder occurs in large or small quantities in all wrought-iron plates; (*a*), (*c*) and (*g*) are not serious, but (*e*) is. Mills will generally guarantee tensile strength of 46,000 to 50,000 lb. per sq. in. (one mill only 42,000) from specimens cut longitudinally; no guarantee can be secured for specimens cut transversely, but strength will be about 40,000. Elastic limit is claimed to be more than 50 per cent. of ultimate strength; some tests have given 57 to

Table 75.—Properties of Riveted Lap Joints for Steel Pipes and Tanks.— *Continued*

Thickness of plate, in.	Number of rows	⅝-in. rivets Eff.	⅝-in. rivets Pitch	⅝-in. rivets Sec.	¾-in rivets Eff.	¾-in rivets Pitch	¾-in rivets Sec.	⅞-in rivets Eff.	⅞-in rivets Pitch	⅞-in rivets Sec.
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
½	1	245	188	122	294	225	147	344	263	172
	2	491	188	245	589	225	294	644	280	322
	3	648	213	324	695	286	347	729	371	364
	4	711	259	355	752	353	376	783	461	391
17/32	1	232	188	123	277	225	147	323	263	172
	2	465	188	247	554	225	294	630	270	335
	3	633	205	336	682	275	362	718	355	381
	4	698	248	371	740	337	393	772	441	410
9/16	1				262	225	147	305	263	172
	2				524	225	295	611	263	344
	3				669	263	376	708	340	398
	4				729	322	410	762	421	429
19/32	1				248	225	147	288	263	171
	2				496	225	294	577	263	343
	3				657	255	390	695	327	413
	4				719	311	427	752	402	447
⅝	1				236	225	147	275	263	172
	2				471	225	294	550	263	344
	3				645	247	403	685	316	428
	4				708	300	443	742	389	464
21/32	1				224	225	147	262	263	172
	2				449	225	295	524	263	344
	3				634	239	416	671	303	440
	4				698	290	458	731	371	480
	5				743	340	488	772	439	507
11/16	1				214	225	147	250	263	172
	2				428	225	294	500	263	344
	3				623	232	428	662	297	455
	4				688	280	473	725	362	498
	5				734	328	505	766	428	527

Thickness of plate, in.	Number of rows	¾-in. rivets Eff.	¾-in. rivets Pitch	¾-in. rivets Sec	⅞-in rivets Eff.	⅞-in rivets Pitch	⅞-in rivets Sec.
①	②	③	④	⑤	⑥	⑦	⑧
23/32	1	205	225	147		263	172
	2	410	225	294	478	263	344
	3	613	226	441	654	288	470
	4	678	272	487	715	351	514
	5	725	318	521	758	414	545
¾	1	196	225	147	229	263	172
	2	393	225	295	458	263	344
	3	589	225	442	644	280	483
	4	669	264	502	707	340	530
	5	716	308	537	751	400	563
25/32	1	189	225	148	220	263	172
	2	377	225	295	440	263	344
	3	566	225	442	635	273	496
	4	660	257	516	698	331	545
	5	708	300	553	742	389	580
13/16	1	181	225	147	211	263	171
	2	363	225	295	423	263	344
	3	544	225	442	624	267	507
	4	651	250	529	689	322	560
	5	700	291	569	734	378	596
27/32	1	174	225	147	204	263	172
	2	349	225	294	407	263	344
	3	523	225	441	611	263	516
	4	642	245	542	681	314	575
	5	692	284	584	728	367	614
⅞	1	168	225	147	196	263	172
	2	337	225	295	393	263	344
	3	505	225	442	589	263	515
	4	634	239	555	674	306	590
	5	684	277	599	720	358	630

Plates ½ to ⅞ in. thick; rivets, ½, ⅝, ¾, ⅞. These tables have been calculated on following assumptions: If value of plate in tension = 100, value of rivet in shear = 75, value of rivet in bearing = 150 Diam. of rivet hole ⅛ in. greater than nominal diam of rivet. Thickness of plate and number of rows of rivets given in columns 1 and 2. Under each rivet size are given efficiency of joint, pitch of rivets and effective section. Efficiency is in scale of 1000. Pitch, inches; decimal point, two figures from right. Section, sq in; decimal point, three figures from right. Effective section per inch is area per inch × efficiency = thickness × eff.

74 per cent. Elongation of 10 per cent. in 8 in. can be specified; there are test results up to 17 per cent. Reduction in area may be required to be 20 per cent. Mills object to having method of manufacture specified. For allowed variation in thickness of plate 3 per cent. may be specified. Dimensions of plates should be given with tolerance of $\frac{5}{16}$ in. in width and ⅜ in. in length. Price of wrought-iron plates when basic price of steel was $0.017 was $0.025 per lb. at mill. Higher price of wrought-iron plates is due to large percentage of condemned plates (about 15 per cent.) and smaller demand.

Durability of Wrought-iron Plates. Wrought-iron pipes in California 40 years or more old are in good condition. Tests of wrought-iron and steel cups filled with water and cinders showed loss by corrosion of 40 per cent. of former and 89 per cent. of latter. Sheet metal mills call for wrought-iron plates, rather than their own steel plates, for annealing boxes. Wrought-iron roofing has lasted 13 years alongside steel which went in 3 years. Iron smoke stacks last 15 years, steel 3. Cost of riveted wrought-iron pipe is about 1.33 times price of steel. Life of steel pipe may be assumed at 35 years.

Table 76. Properties of Double Butt Strap Riveted Joints for Steel Pipes and Tanks

Chicago Bridge & Iron Works

Thickness of plate	No. of rows	3/4" rivets Eff.*	3/4" rivets Pitch	3/4" rivets Sec.*	3/4" rivets Strap	7/8" rivets Eff.*	7/8" rivets Pitch	7/8" rivets Sec.*	7/8" rivets Strap	1" rivets Eff.*	1" rivets Pitch	1" rivets Sec.*	1" rivets Strap
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫	⑬	⑭
1/2	1	500	225	250	5 5/8" × 5/16"	496	263	248	6 1/2" × 5/16"	...			...
	2	720	313	360	9 5/8" × 5/16"	724	362	362	11 1/2" × 3/8"				
	3	794	425	397	14 1/8" × 7/16"	797	494	399	16 1/2" × 1/2"				
	4	837	538	419	22 1/8" × 9/16"	840	625	420	25 1/4" × 5/8"				
17/32	1	500	225	266	5 5/8" × 3/8"	500	263	266	6 1/2" × 3/8"				
	2	720	313	383	9 5/8" × 3/8"	724	362	385	11 1/2" × 3/8"				
	3	794	425	422	14 1/8" × 7/16"	797	494	423	16 1/2" × 1/2"				
	4	837	538	445	22 1/8" × 9/16"	840	625	445	25 1/4" × 5/8"				
9/16	1	500	225	281	5 5/8" × 3/8"	500	263	281	6 1/2" × 3/8"				
	2	720	313	405	9 5/8" × 3/8"	724	362	407	11 1/2" × 3/8"				
	3	794	425	447	14 1/8" × 7/16"	797	494	447	16 1/2" × 1/2"				
	4	837	538	471	22 1/8" × 11/16"	840	625	473	25 1/4" × 5/8"				
19/32	1	496	225	295	5 5/8" × 3/8"	500	263	295	6 1/2" × 3/8"				
	2	718	311	426	9 5/8" × 3/8"	724	362	430	11 1/2" × 3/8"				
	3	793	422	471	14 1/8" × 7/16"	797	494	473	16 1/2" × 1/2"				
	4	837	533	497	21 3/8" × 11/16"	840	625	500	25 1/4" × 5/8"				
5/8	1	471	225	295	5 5/8" × 3/8"	500	263	312	6 1/2" × 3/8"				
	2	707	299	442	9 5/8" × 3/8"	724	362	453	11 1/2" × 3/8"				
	3	784	405	490	14 1/8" × 7/16"	797	494	498	16 1/2" × 1/2"				
	4	829	511	518	21 3/8" × 11/16"	840	625	525	25 1/4" × 5/8"				
21/32	1	449	225	295	5 5/8" × 7/16"	500	263	328	6 1/2" × 7/16"				
	2	698	289	458	9 5/8" × 7/16"	724	362	475	11 1/2" × 7/16"				
	3	764	371	501	13 3/4" × 7/16"	797	494	523	16 1/2" × 1/2"				
	4	822	491	540	20 5/8" × 1/2"	840	625	552	25 1/4" × 5/8"				
	5	852	592	559	29 5/8" × 5/8"			...	...				
11/16	1	428	225	295	5 5/8" × 7/16"	500	263	344	6 1/2" × 7/16"				
	2	688	280	473	9 5/8" × 7/16"	724	362	498	11 1/2" × 7/16"				
	3	767	376	527	13 5/8" × 7/16"	797	494	548	16 1/2" × 1/2"				
	4	815	473	560	19 7/8" × 1/2"	840	625	578	25 1/4" × 5/8"				
	5	846	569	582	28 5/8" × 9/16"	...	...						
23/32	1	409	225	295	5 5/8" × 7/16"	478	263	343	6 1/2" × 7/16"				
	2	678	272	487	9 5/8" × 7/16"	715	351	514	11 1/2" × 7/16"				
	3	759	364	546	13 5/8" × 7/16"	790	476	568	16 1/2" × 1/2"				
	4	808	456	581	19 7/8" × 1/2"	834	602	600	24 1/2" × 5/8"				
	5	840	548	604	27 5/8" × 9/16"	...		...	...				
3/4	1	392	225	295	5 5/8" × 7/16"	458	263	344	6 1/2" × 7/16"	500	300	375	7 1/2" × 7/16"
	2	668	264	501	9 5/8" × 7/16"	706	341	530	11 1/2" × 7/16"	730	412	548	13 1/2" × 7/16"
	3	751	352	564	13 5/8" × 7/16"	783	461	587	16 1/2" × 1/2"	800	562	600	19 1/2" × 9/16"
	4	801	440	601	19 1/8" × 7/16"	828	581	621	24 1/2" × 5/8"	842	713	632	29 1/4" × 3/4"
	5	834	529	626	26 5/8" × 9/16"	857	701	643	34 1/2" × 3/4"	...	...	...	...
25/32	1				...	440	263	344	6 1/2" × 1/2"	500	300	391	7 1/2" × 1/2"
	2				...	698	331	545	11 1/2" × 1/2"	730	412	570	13 1/2" × 1/2"
	3				...	776	446	606	16 1/2" × 1/2"	800	562	625	19 1/2" × 9/16"
	4				...	819	552	640	23" × 9/16"	842	713	658	29 1/4" × 3/4"
	5				...	852	677	666	33 1/2" × 11/16"	...	...	...	...
13/16	1				...	423	263	344	6 1/2" × 1/2"	483	300	392	7 1/2" × 1/2"
	2					690	322	561	11 1/2" × 1/2"	720	402	585	13 1/2" × 1/2"
	3					769	433	625	16 1/2" × 1/2"	795	548	646	19 1/2" × 9/16"
	4					816	544	663	23" × 9/16"	838	693	681	28 1/2" × 11/16"
	5				...	847	655	688	32 1/2" × 9/16"	...	...	...	...

* See footnote on p 301.

Table 76. Properties of Double Butt Strap Joints.—*Continued*

Thickness of plate	No of rows	⅞″ rivets Eff.	⅞″ rivets Pitch	⅞″ rivets Sec	⅞″ rivets Strap	1″ rivets Eff	1″ rivets Pitch	1″ rivets Sec	1″ rivets Strap
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩
27/32	1	407	263	344	6½″× ½″	466	300	393	7½″× ½″
	2	682	314	576	11½″× ½″	713	392	602	13½″× ½″
	3	762	421	646	16½″× ½″	788	532	665	19½″× 9/16″
	4	811	528	684	22¼″× 9/16″	832	671	702	27½″× 11/16″
	5	842	635	711	32½″× 11/16″				
7/8	1	393	263	344	6½″× ½″	449	300	393	7½″× ½″
	2	674	307	590	11½″× ½″	705	382	617	13½″× ½″
	3	756	409	662	16½″× ½″	782	516	684	19½″× 9/16″
	4	805	512	704	22¼″× 9/16″	827	651	724	27 ″× 11/16″
	5	837	615	732	31½″× ⅝″				
29/32	1	379	263	344	6½″× 9/16″	433	300	393	7½″× 9/16″
	2	665	299	603	11½″× 9/16″	700	372	634	13½″× 9/16″
	3	750	400	680	16½″× 9/16″	776	503	703	19½″× 9/16″
	4	800	498	725	21½″× 9/16″	822	633	746	27 ″× ⅝″
	5	833	598	755	30½″× ⅝″				
15/16	1	367	263	344	6½″× 9/16″	419	300	393	7½″× 9/16″
	2	657	292	616	11½″× 9/16″	690	362	647	13½″× 9/16″
	3	743	389	696	16½″× 9/16″	769	488	721	19½′ × 9/16″
	4	794	485	744	21½″× 9/16″	817	614	766	26½″× ⅝″
	5	829	587	777	30½″× ⅝″				
31/32	1	355	263	344	6½′ × 9/16″	408	300	393	7½″× 9/16″
	2	650	286	630	11½″× 9/16″	684	356	663	13½″× 9/16″
	3	737	380	714	16½″× 9/16″	764	477	740	19½″× 9/16″
	4	789	473	764	21½″× 9/16″	813	599	785	25½″× ⅝″
	5	823	565	797	29½″× 9/16″				
1	1	344	263	344	6½″× 9/16″	393	300	393	7½″× 9/16″
	2	644	280	644	11½″× 9/16″	677	348	677	13½″× 9/16″
	3	730	371	730	16½″× 9/16″	759	466	759	19½″× 9/16″
	4	783	461	783	21½″× 9/16″	807	584	807	25½″× ⅝″
	5	818	551	818	28½″× 9/16″	840	701	840	35½″× ¾″
	6	844	641	844	39″ × 11/16″				
1 1/32	1	333	263	344	6½″× ⅝″	381	300	393	7½″× ⅝″
	2	634	275	654	11½″× ⅝″	670	341	691	13½″× ⅝″
	3	724	362	749	16½″× ⅝″	753	455	777	19½ ′× ⅝″
	4	777	450	801	21½″× ⅝″	803	570	828	25½″× ⅝″
	5	809	534	834	28½″× ⅝″	835	683	861	35½″× 11/16″
	6	840	625	866	37¼″× ⅝″				
1 1/16	1	323	263	344	6½″× ⅝″	370	300	393	7½″× ⅝″
	2	630	270	670	11½ ′× ⅝″	663	334	704	13½″× ⅝″
	3	718	355	763	16½″× ⅝″	747	445	794	19½″× ⅝″
	4	773	440	821	21½″× ⅝″	800	556	850	25½″× ⅝″
	5	810	525	861	27½″× ⅝″	831	667	883	34½″× 11/16″
	6	836	609	888	36½″× ⅝″				
1 3/32	1	315	263	344	6½″× ⅝″	360	300	393	7½″× ⅝″
	2	622	265	680	11½″× ⅝″	657	328	719	13½″× ⅝″
	3	712	347	779	16½″× ⅝″	742	436	812	19½″× ⅝″
	4	767	430	839	21½″× ⅝″	793	544	868	25½″× ⅝″
	5	807	518	883	27½″× ⅝″	828	651	905	33½″× 11/16″
	6	832	595	910	36½″× ⅝″				
1⅛	1	305	262	344	6½″× ⅝″	346	300	393	7½″× ⅝″
	2	610	262	688	11½″× ⅝″	650	322	731	13½″× ⅝″
	3	708	341	797	16½″× ⅝″	736	427	828	19½″× ⅝″
	4	762	421	857	21½″× ⅝″	788	531	886	25½″× ⅝″
	5	800	501	900	26½″× ⅝″	823	636	926	33½″× ⅝″
	6	828	581	932	36½″× ⅝″				
1 5/32	1	297	263	344	6½″× 11/16″	339	300	393	7½″× 11/16″
	2	594	263	688	11½″× 11/16″	644	316	745	13½″× 11/16″
	3	700	334	810	16½″× 11/16″	731	418	845	19½″× 11/16″
	4	757	412	875	21½″× 11/16″	784	520	906	25½″× 11/16″
	5	796	491	920	26½″× 11/16″	819	621	947	32½″× 11/16″
	6	824	568	953	35¼″× 11/16″				
1 3/16	1	290	263	344	6½″× 11/16″	331	300	393	7½″× 11/16″
	2	580	263	688	11½″× 11/16″	638	311	758	13½″× 11/16″
	3	694	327	824	16½″× 11/16″	726	410	862	19½″× 11/16″
	4	752	404	893	21½″× 11/16″	779	509	925	25½″× 11/16″
	5	792	480	941	26½″× 11/16″	815	609	968	31½″× 11/16″
	6	820	556	974	34 ″× 11/16″	841	707	992	42½″× ¾″
	7	842	632	1000	45½″× 11/16″				

Table 76. Properties of Double Butt Strap Joints.—*Continued*

Thickness of plate	No. of rows	$\frac{7}{8}''$ rivets				1″ rivets			
		Eff.	Pitch	Sec.	Strap	Eff.	Pitch	Sec.	Strap
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩
	1	282	262	344	$6\frac{1}{2}''\times\frac{11}{16}''$	322	300	393	$7\frac{1}{2}''\times\frac{11}{16}''$
	2	564	262	688	$11\frac{1}{2}''\times\frac{11}{16}''$	632	306	770	$13\frac{1}{2}''\times\frac{11}{16}'$
	3	690	322	842	$16\frac{1}{2}''\times\frac{11}{16}''$	721	403	879	$19\frac{1}{2}''\times\frac{11}{16}'$
$1\frac{7}{32}$	4	747	396	913	$21\frac{1}{2}''\times\frac{11}{16}''$	775	500	945	$25\frac{1}{2}''\times\frac{11}{16}''$
	5	787	470	959	$26\frac{1}{2}''\times\frac{11}{16}''$	811	596	988	$31\frac{1}{2}''\times\frac{11}{16}''$
	6	816	544	993	$34''\times\frac{11}{16}''$	838	693	1021	$42\frac{1}{2}''\times\frac{11}{16}''$
	7	838	618	1021	$44''\times\frac{11}{16}''$	....	..	.....	....
	1	275	262	344	$6\frac{1}{2}''\times\frac{11}{16}''$	314	300	393	$7\frac{1}{2}''\times\frac{11}{16}''$
	2	550	262	688	$11\frac{1}{2}''\times\frac{11}{16}''$	628	300	786	$13\frac{1}{2}''\times\frac{11}{16}''$
	3	684	317	855	$16\frac{1}{2}''\times\frac{11}{16}''$	714	394	893	$19\frac{1}{2}''\times\frac{11}{16}''$
$1\frac{1}{4}$	4	748	389	935	$21\frac{1}{2}''\times\frac{11}{16}''$	770	488	963	$25\frac{1}{2}''\times\frac{11}{16}''$
	5	783	461	979	$26\frac{1}{2}''\times\frac{11}{16}''$	807	583	1009	$31\frac{1}{2}''\times\frac{11}{16}''$
	6	812	533	1015	$34''\times\frac{11}{16}''$	831	677	1039	$41\frac{1}{2}''\times\frac{11}{16}''$
	7	835	605	1044	$42\frac{1}{2}''\times\frac{11}{16}''$	...	.	.	

Shop Calking. Lap joints are calked all around, inside and outside; butt joints on the outside only. (See also p. 328 for "Field Calking.")

Shop Testing. To be sure of tight work, every shop length should be tested before coating. This can be done rapidly and cheaply by means of special heads with rubber or similar gaskets, requiring no riveting. Use flat-head, soft-rubber plugs in empty rivet holes. Same test heads can be used in field tests. One advantage of shop test is that refilling of the trench between field joints can be done at once, thus protecting the pipe from extreme expansion and contraction. Making the pipe tight is cheaper in the shop than in the field.

DESIGN OF RIVETED JOINTS, STEEL PIPE SIPHONS, CATSKILL AQUEDUCT*

Treatment of Steel Pipe Siphons. Pipes crossing some small valleys are of steel plate, diam. approximately 9.5 and 11 ft., lined with Portland cement mortar having a minimum thickness of 2 in., and jacketed with concrete having a minimum thickness of 6 in. After laying and calking, the pipes were filled with water to hydraulic gradient of the adjacent portions of the aqueduct, and while under this pressure were jacketed. After the concrete had attained some strength, the water was drained out and the lining applied. (See also p. 323.)

Assumptions. No allowance was made for corrosion with lined and covered pipes and the nominal was taken as the working thickness of the steel plate. With unlined pipes, the working thickness of steel plate was assumed $t-\frac{1}{16}$ in.

d = nominal diam. of rivet, inches

d_1 = driven diam. of rivet $= d+\frac{1}{16}$ in.

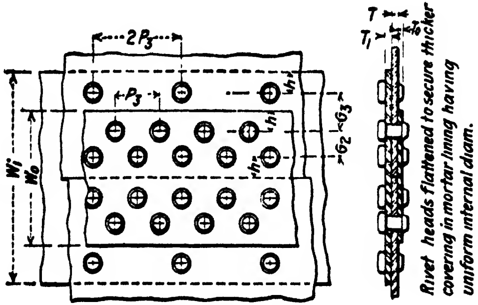
t = thickness of steel plate, inches

p = diagonal pitch of rivets, inches

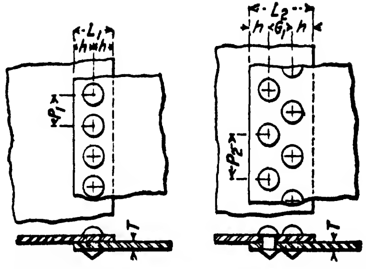
p_1 = longitudinal pitch of rivets, inches

a_1 = length of plate between rivet holes $= p_1 - d_1$

*Following formulas, tables, etc., are given as examples only.



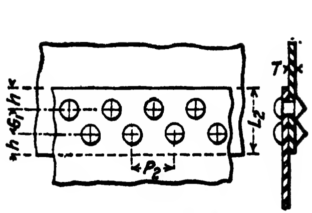
Longitudinal Seam Triple-Riveted Butt.
(81% Efficiency)



Single Riveted Lap. (50% Efficiency)

Double-Riveted Lap.

Circular Seam.



Longitudinal Seam.
Double-Riveted Lap.
(70% Efficiency)

Thickness of Shell	Longitudinal Seam								
	Pitch of Rivets		Gage Distance		Edge Distance	Width of Cover Plates		Thickness of Cover Plates	
	Inner Rows	Outer Rows	Inner Rows	Outer Rows		Outer	Inner	Outer	Inner
T	P_3	$2P_3$	G_2	G_3	h	W_0	W_1	T_0	T_i
1/2"	3.58"	7.16"	1 11/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	7/16"	9/16"
9/16"	4.00"	8.00"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	5/8"
11/16"	3.82"	7.64"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	6/8"
3/4"	3.58"	7.16"	1 11/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	1/2"	6/8"

Thickness of Shell	Edge Distance	Single Riveted			Double Riveted			
		Number of Riv	Approx. Pitch	Lap	No of Rivets each Row	Approx. Pitch	Gage Distance	Lap
T	h		P_1	L_1		P_2	G_1	L_2
7/16"	1 5/8"	144	2.5"	3 1/4"	103	3.50"	1 15/16"	5 3/16
1/2"	1 5/8"	Not Used			116	3.11"	2"	5 1/4
9/16"	1 5/8"	"	"		123	2.95"	2 1/16	5 5/16
11/16"	1 5/8"	"	"		139	2.61"	2 3/16	5 7/16
3/4"	1 5/8"	"	"		145	2.50"	2 3/16	5 7/16

Thickness of Shell	Longitudinal Seam			
	Pitch of Rivets	Gage Distance	Edge Distance	Lap
T	P_2	G_1	h	L_2
7/16"	3.50"	1 15/16"	1 5/8"	5 3/16
1/2"	3.11"	2"	1 5/8"	5 1/4

All pipe to have alternate inside and outside courses (i.e. not taper).
All rivets 1 in. diam.
Inside diameter of inside course = 9' 6"

Assumed tensile stress on net section of plate 15,000 lbs per sq. in.
" bearing " " rivets and plate 22,500 " " " "
" shearing " " plate 10,000 " " " "
" " " " rivets-single 9,000 " " " "
" " " " rivets-double 8,000 " " " "
All Computations based on diam. of driven rivets - $1\frac{1}{16}$"

FIG. 157.—Steel pipe siphons, riveted joints, Catskill aqueduct.

g = gage distance, inches (*i.e.*, distance between two rows of rivets in a plate)
h = edge distance, inches, from center of rivet; h', from edge of rivet hole
f_t = unit tensile stress on steel plate
f_c = unit bearing stress on rivets and plates
f_s = unit shearing stress on steel plate
f'_s = unit shearing stress on rivets in single shear
f''_s = unit shearing stress on rivets in double shear
ϵ = efficiency of joint
k = allowance for clearance between rivet heads = $\frac{1}{2}''$

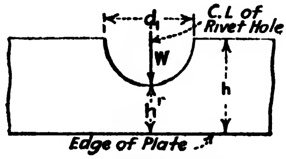


FIG. 158.—Steel pipe, edge distance. (Table 77.)

Working stresses assumed, in lbs. per sq. in., were $f_t = 15{,}000$; $f_c = 22{,}500$; $f_s = 10{,}000$; $f'_s = 9000$; $f''_s = 8000$.

Edge distance. *Theoretical edge distance,* $h = d_1\left(0.5 + 0.595\sqrt{\frac{d_1}{t}}\right)$.

Table 77. Steel Pipe. Edge Distance, from Consideration that Distance from Center of Rivet to Edge of Plate shall be $1\frac{1}{2}$ times Diam. of Driven Rivet

(All dimensions in inches)

Thickness of plate, t	Diam. of driven rivet, d_1	$\frac{d_1}{t}$	Edge distance: theoretical	Edge distance: from $h = 1\frac{1}{2}d_1$	Edge distance: used	Lap of circular seams, single riveted
$\frac{7}{16}$	$1\frac{1}{16}$	2 43	1 52	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{7}{16}$	$1\frac{1}{8}$	2 57	1 64	1 69	$1\frac{11}{16}$	$3\frac{3}{8}$
$\frac{1}{2}$	$1\frac{1}{16}$	2 13	1 45	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{1}{2}$	$1\frac{1}{4}$	2 50	1 80	1 88	$1\frac{7}{8}$	$3\frac{3}{4}$
$\frac{9}{16}$	$1\frac{1}{16}$	1 89	1 40	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{9}{16}$	$1\frac{3}{16}$	2 11	1 62	1 78	$1\frac{13}{16}$	$3\frac{5}{8}$
$\frac{5}{8}$	$1\frac{1}{16}$	1 70	1 35	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{5}{8}$	$1\frac{1}{8}$	1 80	1 45	1 69	$1\frac{11}{16}$	$3\frac{3}{8}$
$\frac{5}{8}$	$1\frac{3}{16}$	1 90	1 57	1 78	$1\frac{13}{16}$	$3\frac{5}{8}$
$\frac{11}{16}$	$1\frac{1}{16}$	1 54	1 31	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{11}{16}$	$1\frac{3}{16}$	1 73	1 52	1 78	$1\frac{13}{16}$	$3\frac{5}{8}$
$\frac{11}{16}$	$1\frac{1}{4}$	1 82	1 64	1 88	$1\frac{7}{8}$	$3\frac{3}{4}$
$\frac{3}{4}$	$1\frac{1}{16}$	1 42	1 28	1 59	$1\frac{5}{8}$	$3\frac{1}{4}$
$\frac{3}{4}$	$1\frac{3}{16}$	1 58	1 48	1 78	$1\frac{13}{16}$	$3\frac{5}{8}$

Gage Distance and Diagonal Pitch. I. The gage distance, g, required for equal diagonal and longitudinal strength, *i.e.*, make resistance to tear and shear along $k - f$ equal to $\frac{1}{2}$ resistance to tear along $n - e$. See Fig. 159.

To find g when p_1, d_1 and t are known.

Resistance to rupture along $k' - f'$ is 96 per cent. that along $k - f$, and is a minimum for the pitch and gage distance assumed. Therefore resistance along $k - f$ must be made equal to $\frac{100}{96} \cdot \frac{a_1}{2}$

$$\left(1 - \frac{2d_1}{\sqrt{p_1^2 + 4g^2}}\right)^2 - \frac{\left[\frac{300}{96}(p_1 - d_1)\right]^2}{9p_1^2 + 16g^2} = 0 \quad .$$

Find the value of g to satisfy this equation for equal diagonal and longitudinal strength.

II. Gage distance for clearance, diam. of rivet head = $2(d_1 - \frac{1}{16}'') = 2d.$

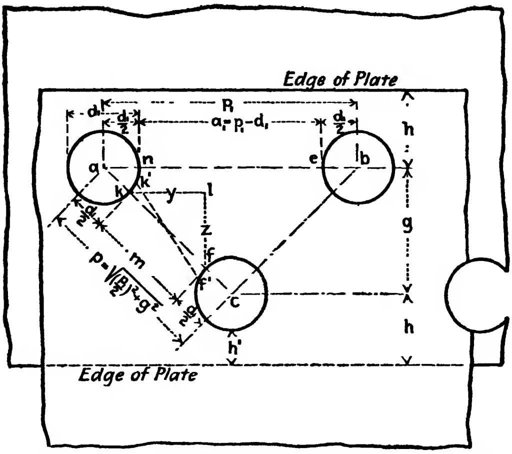


Fig. 159.—Gage distance and diagonal pitch.

Hence

$$p = 2(d_1 - \tfrac{1}{16}'') + k = 2d + \tfrac{1}{2}''$$

$$g = \sqrt{p^2 - \left(\frac{p_1}{2}\right)^2} = \sqrt{(2d + \tfrac{1}{2}'')^2 - \left(\frac{p_1}{2}\right)^2}$$

III. Gage distance for $\frac{1}{4}$ in. greater net diagonal than longitudinal distance.

$$i.e.,\ 2p = p_1 + d_1 + \tfrac{1}{4}''$$

$$p = \frac{p_1 + d_1}{2} + \tfrac{1}{8}''$$

$$g = \sqrt{\left(\frac{p_1 + d_1}{2} + \tfrac{1}{8}''\right)^2 - \left(\frac{p_1}{2}\right)^2}$$

Table 78. Steel Pipe—Riveted Joints—Gage Distance, Inches

Thickness of plate and type of joint	Diam of driven rivet	Long pitch*	Gage distance for $\frac{1}{2}''$ clearance	Gage distance for $\frac{1}{4}''$ greater net diag dist.	Gage for equal diag. and long. strength	Gage dist. used
$\frac{7}{16}L$	$1\frac{1}{16}$	3 50	1 79	1 65	1 89	$1\frac{15}{16}$
$\frac{1}{2}L$	$1\frac{1}{16}$	3 11	1 96	1 57	1 76	2
$\frac{1}{2}B$	$1\frac{1}{16}$	3 58	1 75	1 67	1 92	$1\frac{15}{16}$
$\frac{9}{16}B$	$1\frac{1}{16}$	4 00	1 50	1 75	2 05	$2\frac{1}{16}$
$\frac{5}{8}B$	$1\frac{1}{16}$	4 09	1 44	1 77	2 09	$2\frac{1}{8}$
$\frac{11}{16}B$	$1\frac{1}{16}$	3 82	1 61	1 71	2 01	$2\frac{1}{16}$
$\frac{3}{4}B$	$1\frac{1}{16}$	3 58	1 75	1 67	1 92	$1\frac{15}{16}$
* $\frac{9}{16}B$	$1\frac{3}{16}$	4 00	1 89	1 84	2 16	$2\frac{3}{16}$ *
* $\frac{5}{8}B$	$1\frac{3}{16}$	4 00	1 89	1 84	2 16	$2\frac{3}{16}$ *
* $\frac{11}{16}B$	$1\frac{3}{16}$	4 00	1 89	1 84	2 16	$2\frac{3}{16}$ *
* $\frac{3}{4}B$	$1\frac{3}{16}$	4 00	1 89	1 84	2 16	$2\frac{3}{16}$ *
$\frac{9}{16}B$	$1\frac{3}{16}$	3 55	2 10	1 75	1.99	$2\frac{1}{8}$
$\frac{11}{16}B$	$1\frac{3}{16}$	3 12	2 27	1 66	.. ..	$2\frac{1}{4}$
$\frac{3}{4}B$	$1\frac{3}{16}$	2 96	2 315	1 63	.... .	$2\frac{3}{8}$

* Longitudinal pitch except lines 8 to 11 such as to give maximum practicable efficiency with diam of rivet in Col 2. For lines 8 to 11 inclusive, longitudinal pitch taken = 8 times thickness of outer cover plate, which is $\frac{1}{2}$ in. thick.

Single-riveted Lap Joint. (See page 305.)
Methods of failure: a. Tearing plate. b. Shearing rivets.
For length of joint $= p_1$:

Resistance to tearing $= (p_1 - d_1)\,tf_t$;

to shear $= \dfrac{\pi d_1^2}{4} f'_s$;

of solid plate $= p_1 t f_t$.

For equal strength against tearing and shearing, $p_1 = \dfrac{d_1}{1-\epsilon}$, unless limited by clearance, when $p \nless 2d + \frac{1}{2}$ in. $\quad \epsilon = \dfrac{0.471\dfrac{d_1}{t}}{1 + 0.471\dfrac{d_1}{t}}$

The constant in the ϵ formula depends on the value of f'_s and f_t used, 0.471 being replaced by $\dfrac{\pi}{4} \times \dfrac{f'_s}{f_t}$.

Table 79. Steel Pipe—Single-riveted Lap Joints; Efficiency, Pitch and Strength

Thickness of plate, t	Diam. of driven rivet = diam. of hole, d_1	Lap of joint $=2\times$ edge distance $=2h$ (See Table 77)	Efficiency % Maximum $\epsilon = \dfrac{0.471\frac{d_1}{t}}{1+0.471\frac{d_1}{t}}$	Efficiency % Developed when pitch is fixed by clearance requirement	Pitch, p_1, inches for max eff $p_1 = \dfrac{d_1}{1-\epsilon}$	Pitch, p_1, inches for clearance requirements $p_1 \nless 2d + \frac{1}{2}''$	Strength, per in. of joint, using p_1 from previous column: Shear, lbs.	Strength, per in. of joint, using p_1 from previous column: Tension, lbs
$\frac{7}{16}$	1	3	51 8	45 4	2.07	$2\frac{3}{8}$	2980	3800
	$1\frac{1}{16}$	$3\frac{1}{4}$	53.3	48.6	2 28	$2\frac{1}{2}$	3190	3770
	$1\frac{1}{8}$	$3\frac{3}{8}$	54.7	52 0	2 48	$2\frac{5}{8}$	3410	3750
	$1\frac{3}{16}$	$3\frac{5}{8}$	56 1	55.3	2 70	$2\frac{3}{4}$	3630	3740
	$1\frac{1}{4}$	$3\frac{3}{4}$	57.4	56.5	2 94	$2\frac{7}{8}$	3840	3710
$\frac{1}{2}$	1	3	48 4	39.7	1 94	$2\frac{3}{8}$	2980	4340
	$1\frac{1}{16}$	$3\frac{1}{4}$	50 0	42.5	2 12	$2\frac{1}{2}$	3190	4310
	$1\frac{1}{8}$	$3\frac{3}{8}$	51 5	45.5	2 32	$2\frac{5}{8}$	3410	4290
	$1\frac{3}{16}$	$3\frac{5}{8}$	52 8	48.4	2 51	$2\frac{3}{4}$	3630	4270
	$1\frac{3}{4}$	$3\frac{3}{4}$	54 1	51.2	2 72	$2\frac{7}{8}$	3840	4240

Diam. of rivet before driving $= d_1 - \frac{1}{16}$.

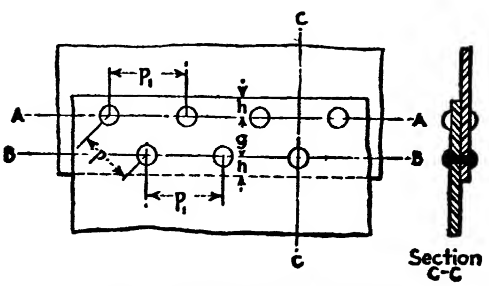


Fig. 160.—Double-riveted lap joint.

Double-riveted Lap Joint. Methods of failure: a. Tearing plate along AA or BB. b. Shearing all rivets. For length of joint $= p_1$:

Resistance to tearing along AA or $BB = (p_1 - d_1)tf_t$; to shearing $= \dfrac{2\pi d_1^2}{4} f'_s = \dfrac{\pi d_1^2}{2} f'_s$;

of solid plate $= p_1 t f_t$.

$p_1 = \dfrac{d_1}{1-\epsilon}$ unless limited by

clearance, when $p \not< 2d + \frac{1}{2}''$

$$\epsilon = \frac{0.943 \dfrac{d_1}{t}}{1 + 0.943 \dfrac{d_1}{t}}.$$ The constant 0.943 is for $f'_t = 15{,}000, f'_s = 9000$; for other values it becomes $\dfrac{\pi}{4}\dfrac{f'_s}{f_t}$

When $\epsilon = 70\%, d_1 = 2.472t, p_1 = \dfrac{d_1}{0.3}$

Table 80. Steel Pipe—Double-riveted Lap Joints. Efficiency and Pitch

Thickness, t, and type of joint	Diam. of driven rivet $= d + \frac{1}{16}''$	$\epsilon = 0.943\frac{d_1}{t} \div (1 + 0.943\frac{d_1}{t})$ %	Pitch $p_1 = \frac{d_1}{1-\epsilon}$	Pitch $p_1 \not> 8t$	Thickness, t, and type of joint	Diam. of driven rivet $= d + \frac{1}{16}''$	$\epsilon = 0.943\frac{d_1}{t} \div (1 + 0.943\frac{d_1}{t})$ %	Pitch $p_1 = \frac{d_1}{1-\epsilon}$	Pitch $p_1 \not> 8t$
$\frac{7}{16}L$	$\frac{15}{16}$	66 9	2 83	3 50	$\frac{1}{2}L$	1	65 4	2.89	4 00
	1	68 3	3 16			$1\frac{1}{16}$	66 7	3 19	
	$1\frac{1}{16}$	69 6	3.50			$1\frac{1}{8}$	67 9	3 50	
	$1\frac{1}{8}$	70 8	3.85			$1\frac{3}{16}$	69 2	3 85	
	$1\frac{3}{16}$	71 9	4 23			$1\frac{1}{4}$	70 2	4 19	
	$1\frac{1}{4}$	72 9	4 61						

Table 81. Steel Pipe—Double-riveted Lap Joints, Longitudinal Seam Efficiency, Pitch and Strength*

(A) Joints designed to give maximum practicable efficiency using rivets up to $1\frac{3}{16}$ in.

(All dimensions in inches)

Thickness of shell t	Diam driven rivet = diam. of hole d_1	Lap of circular seam = 2 × edge dist.	2 × dist from edge of pl at ○ joint to ctr. of nearest rivet in long. seam $2a$†	Dist betw'n ctrs of end rivets in long seam	Approx. pitch for max. eff. p_1	Apprx. no. of spaces	No of rivets in joint: Outer row	No of rivets in joint: Inner row	Pitch of rivets each row p_1	Strength per 1 in of joint, lbs.: Shear	Strength per 1 in of joint, lbs.: Tension	Strength per 1 in of joint, lbs.: Required for 70% efficiency	Eff developed %
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
$\frac{7}{16}$	$1\frac{1}{8}$	$3\frac{3}{8}$	$\not<3\frac{1}{8}$	$\not>83\frac{1}{2}$	3 85	22	..	...	....	...	....	4590	
						22	23	22	3 80	4710	4620		
			3 245	83.38					3 79	4720	4615		70 3
									3 78	4730	4610		
$\frac{1}{2}$	$1\frac{1}{4}$	$3\frac{3}{4}$	$\not<3\frac{3}{8}$	$\not>82\frac{7}{8}$	4 19	20	..	.	..			5250	
						20	21	20	4 15	5320	5240		
			3 45	82 80		.	.	.	4 14	5330	5230		69 7
									4 13	5340	5220		

Course is an outer course If longitudinal joints are double-riveted lap, using staggered riveting, that joint is best, which, (a) when course is an outside one, has one more rivet in line next edge of outside plate; (b) when course is an inner one, has one more rivet in line next edge of inner plate.

Diam. of rivet before driving, $1\frac{1}{16}$ in and $1\frac{3}{16}$ in respectively. * Length c. to c of circular seams = 7 ft. 6 in † See footnote at end of Table 81, p. 310.

Table 81.—*Continued*

(B) Joints designed to give maximum practicable efficiency using 1-in. rivets only

①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫	⑬	⑭
$\frac{7}{16}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$†	≯84	3.50	24	.			. . . . .	. . . .	4590	. . .
						24	25	24	3 51	4545	4570		
			$2\frac{3}{4}$	84	(Used on d'w'gs.)				3 50	4555	4565		69 4
			3 23*	83 52		. .		.	3 48	4580	4560	. .	69 5
$\frac{1}{2}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$†	≯84	3 19	26				. .	. .	5250	.
						26	27	26	3 24	4920	5040	. . . .	
									3 23	4940	5030		
						27	28	27	3 12	5110	4940		
			2 78	83 97	(Used on d'w'gs.)				3 11	5125	4940	. . .	65 9
			3 05*	83 70	. .		.	. . .	3 10	5145	4935	. .	65 8

* Figured on basis that clearance $k = \frac{3}{8}$ in for $d \not> 1\frac{1}{8}$ in It was decided later to make $k = \frac{1}{2}$ in. for all sizes of rivets

† This is as follows (Fig 161) $a = k + d$ Col $4 = 2(k + d)$
$k = \frac{1}{2}$

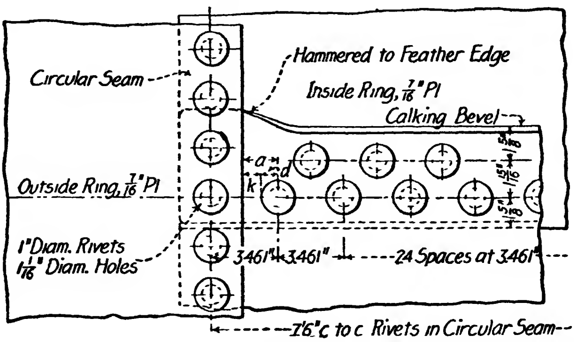


FIG. 161.—Junction of double-riveted lap joint (longitudinal seam) with single-riveted lap joint (circular seam).
(Catskill Aqueduct)

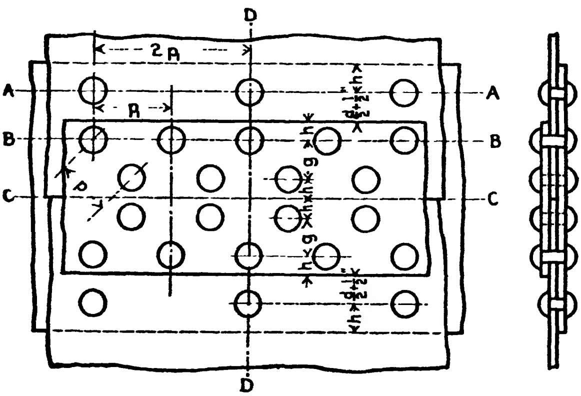


FIG. 162.—Triple-riveted butt joint.

Triple-riveted Butt Joint. Methods of failure: a. Tearing plate along *AA*. b. Tearing plate along *BB* and shearing rivets on *AA*. c. Shearing all rivets on one side of *CC*.

For length of joint $= 2p_1$:

Resistance to tension $= tf_t\ (2p_1 - d_1)$

Resistance to tension + shear $= \frac{\pi d_1^2}{4} f'_s + 2(p_1 - d_1)f_t$

Resistance to shear $= 8\left(\frac{\pi d_1^2}{4}\right)f''_s + \left(\frac{\pi d_1^2}{4}\right)f'_s = \frac{\pi d_1^2}{4}(8f''_s + f'_s)$

Resistance of solid plate $= 2p_1tf_t$.

Table 82. Steel Pipe—Triple-riveted Butt Joints, Longitudinal Seam*

(All dimensions in inches)

Scheme A—Joints designed to give maximum efficiency using up to $1\frac{3}{16}$-in. rivets and conforming to good practice

Thickness of shell t	Diam. of hole = diam of driven rivet d_1	Lap of ⊙ seam = 2 × edge dist.	2 × dist. from edge of shell pl. at ⊙ seam to ctr of nearest rivet in long seam† $2a$	Dist between ctrs. of end rivets in long. seam	Approx pitch	Approx. no spaces	No. rivets in outer cover plate: Outer row	No. rivets in outer cover plate: Inner row	Pitch of rivets, each row	Strength per 1 in. of joint: Tearing of Pl. at outermost row of rivets of inner cover plate	Strength per 1 in. of joint: Tearing of Pl. at middle row of rivets and shearing of outermost row	Strength per 1 in. of joint: Shearing all rivets	Req'd str. of jt. per in. of jt. for 85% eff. and eff developed
$\frac{1}{2}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$†	≯84	4 00	21	22	21	.	....	..	...	6380
									4 01	6505	6505	8060	
									4 00	6503	6503	8080	86 7%
									3 99	6500	6500	8100	
$\frac{9}{16}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	4 42	19	.		.	.	....	...	7180
							20	19	4 43	7435	7322	7300	
									4 42	7432	7320	7320	86.6%
									4 41	7430	7318	7340	
$\frac{5}{8}$	$1\frac{1}{8}$	$3\frac{3}{8}$	≮$2\frac{7}{8}$	≯$83\frac{3}{4}$	4 52	$18\frac{1}{2}$	19	19	4 53	8220	8036	8010	7980
										8215	8032	8030	85 5%
										8212	8030	8040	
$\frac{11}{16}$	$1\frac{1}{4}$	$3\frac{3}{4}$	≮$3\frac{3}{8}$	≯$82\frac{7}{8}$	5 06	$16\frac{1}{2}$	17	17	5.03	9055	8852	8910	8760
									5.02	9052	8850	8920	85 8%
									5.01	9048	8847	8940	

Scheme B—Joints designed to give maximum efficiency using 1-in rivets only and conforming with good practice

$\frac{1}{2}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	3.56	$23\frac{1}{2}$	..		.				6380
			2 62	84 13			24	24	3 58	6384	6384	9050	85 2%
$\frac{9}{16}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	4 00	21+				..			7180
			2 75	84			22	21	4 00	7320	7190	8090	85 2%
$\frac{5}{8}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	4 09	$20\frac{1}{2}$	.	.			.		7980
							21	21	4 10	8160	7920	7890	
			2 905	83 85	.	.			4 09	8155	7915	7910	84.2%
									4 08	8150	7910	7930	
$\frac{11}{16}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	3 81	22		.				.	8760
			2 71	84 04			23	22	3 82	8880	8485	8470	82 2%
									3 81	8875	8480	8480	
									3 80	8870	8475	8500	
$\frac{3}{4}$	$1\frac{1}{16}$	$3\frac{1}{4}$	≮$2\frac{3}{4}$	≯84	3.60	$23\frac{1}{2}$	.	..	.		...		9560
							24	24	3 58	9590	9030	9050	80 2%
									3.57	9585	9025	9070	

* Length c. to c. of rivets in circular seam = 7 ft. 6 in.

† This is as follows. (Fig. 163). $a = k + d$ *Col.* 4 $= 2(k + d)$. In this table clearance $k = \frac{1}{8}$ in. for d ≯ $1\frac{1}{8}$ in. $k = \frac{1}{4}$ in. for $d > 1\frac{1}{8}$ in.

Table 83. Steel Pipe—Riveted Joints, Butt Straps for Longitudinal Joints—using 1-in. Rivets Only ($1\frac{1}{16}$ in. diam. driven)

Value of 1 rivet in single shear = 7980; double shear = 14,180

(All dimensions in inches)

Thickness of main plate	Longitudinal pitch, see table above, 10th col. from left	Thickness of cover plates required						Thickness of cover plates used	
		I‡ To transmit shear borne by rivets		II By condition that outer ∢ $\frac{1}{16}$ pitch, inner, corresponding to shear		III By condition that outer ∢ $\frac{1}{8}$ pitch; inner, corresponding to shear borne			
		Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner
$\frac{1}{2}$	3 58	0 27	0.34	0 36	0.46	0.45	0.57	$\frac{7}{16}$	$\frac{9}{16}$
$\frac{9}{16}$	4.00	0 29	0.37	0 40	0 51	0.50	0 64	$\frac{1}{2}$	$\frac{5}{8}$
$\frac{5}{8}$	4 09	0 31	0.40	0 41	0 53	0.51	0 66	$\frac{1}{2}$	$\frac{5}{8}$
$\frac{11}{16}$	3 82	0 34	0 44	0 38	0 49	0.48	0 61	$\frac{1}{2}$	$\frac{5}{8}$
$\frac{3}{4}$	3.58	0.37	0.48	0.36	0.46	0.45	0.57	$\frac{1}{2}$	$\frac{5}{8}$

‡ Take $\frac{1}{2}$ in. plate with one repeating section of riveting

Stress carried by outer or inner cover plate is proportional to stress transmitted to each from shear on rivets, and is as follows:

In outer cover plate there are 4 rivets in double shear at $\frac{14,180}{2}$ = 28,360 lbs.

In inner cover plate there are 4 rivets in double shear at $\frac{14,180}{2}$ = 28,360 lbs , 1 rivet in single shear at 7980 Total = 36,340.

Total stress in both plates = 36,340 + 28,360 = 64,700. Portion in outer plate = 28,360 ÷ 64,700 = 0 438; portion in inner plate = 36,340 ÷ 64,700 lbs. = 0.562.

Least strength of joint (with 3 58 in. pitch) = 6380 lbs per in. (Table 82).

Strength for length of one repeating section = 2 × 3 58 × 6380 = 45,700 lbs

In outer cover plate in length 2 × 3 58 in. = 7 16 in , there is 7.16 in. − 2.125 in. = 5 035 in net length. Hence 5 035 × t × 15,000 = 0.438 × 45,700 lbs.

$$t = 0.27 \text{ in.}$$

In inner cover plate 5.035 × t × 15,000 lbs = 0 562 × 45,700 lbs

$$t = 0\ 34''.$$

$\frac{9}{16}$ in plate, strength of joint = 7190 lbs. per in., $\frac{5}{8}$ in. plate, strength of joint = 7910 lbs per in.; $\frac{11}{16}$ in. plate, strength of joint = 8470 lbs. per in.; $\frac{3}{4}$ in. plate, strength of joint = 9025 lbs per in

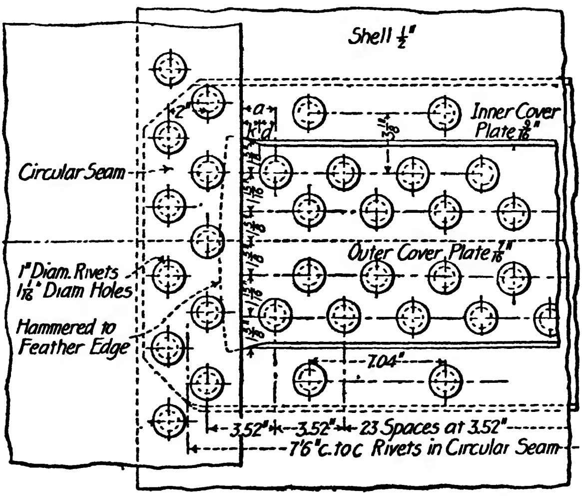


Fig. 163.—Junction of triple-riveted butt joint (longitudinal seam) with double-riveted lap joint (circular seam).

(Catskill Aqueduct)

PLATES FOR STEEL PIPES

Materials. Catskill aqueduct specifications call for steel to be made by basic open-hearth process, plate steel ($\frac{7}{16}$, $\frac{1}{2}$ and $\frac{9}{16}$-in. thickness, for 9 ft. 6 in. to 11 ft. 3 in. diam.) to conform to "flange steel," and rivets to "extra soft steel" requirements of "Standard Specifications for Open-hearth Boiler Plate and Rivet Steel," Am. Soc. Test. Mat., Aug. 16, 1909.

Minimum requirements for plates and rivets.

Tensile strength	Over 50,000 lb. per sq. in.
Elastic limit. . . .	Over 30,000 lb. per sq. in.
Elongation in 8 in.	18 per cent.
Reduction in area	50 per cent.

Steel was preferred to wrought iron because the latter could not be obtained in sufficiently large sheets; strength would be less, hence thickness and cost greater; price per lb. greater; deliveries slow; the evidences of greater resistance to corrosion were not conclusive. In view of present knowledge of corrosion, some authorities claim steel should approximate composition of pure iron. Following analysis has been suggested by one mill: carbon, 0.04 to 0.06 per cent.; manganese, 0.15; phosphorus, 0.01; sulphur, 0.04. Test pieces cut from plates should have elongation of 30 to 34 per cent. in 8 in., and ultimate strength 42,000 to 48,000 lbs. This steel has been successfully rolled into plates.

Mill Inspection of Open-hearth Plate-rolling. (*a*) Chemical analyses should be made from sample drawn off ingot and check sample from drillings from tensile test specimen. (*b*) Physical tests include 3 tensile tests of clippings from plate; a fourth sample should be taken for use if one of the three fails. Gage of plate is judged from thickness of test specimen after shearing. Some steel mills furnish a mill tester, the inspector merely checking readings of the testing machine. Cold and quench bending tests should be witnessed by inspector. Inspector is not required to make surface inspection of plate; this occurs in the fabricating shop, usually.

Thickness of Plate should be determined as follows:* (1) $t = pdf \div 2t_sE$, t = thickness of plate, in.; p = internal pressure, lb. per sq. in.; f = factor of safety; t_s = tensile strength of plate, lb. per sq. in. = (55,000); E = efficiency of joint; d = internal diameter of pipe, in. It is good practice to use 15,000 for $t_s \div f$. Constant, $\frac{1}{16}$ in., is sometimes added to computed thickness as provision against rusting.

Most shops cannot handle plates of greater thickness than 1 in.; one can take $1\frac{1}{2}$ in.; $1\frac{1}{2}$-in. plate in pipes costs about 25 per cent. more per lb. than 1 in., due to necessity of drilling holes after plate is rolled to cylinder, to avoid making holes oblong. Thin pipe is more expensive than medium weight, due to larger number of rivets required.

Plate Lengths. Specify maximum as well as minimum lap. Edges of plates as delivered from mills are not straight; when riveted, lap is sometimes as much as 2 in. in excess.

* Dept. Water Supply, Gas and Electricity, N. Y.

Bending Plate. Good practice requires plates rolled cold to true curve, no heating or hammering being allowed.

Beveling Edges of Plates. Plates are often cut and beveled at one operation by a multiple shearing machine. Edges sometimes are beveled by planing. Many persons prefer bevel edges on all plates $\frac{1}{2}$ in. or more thick, to facilitate calking. On the Catskill aqueduct it was required that the bevel be formed at 45° by planing; 60° is sometimes required. If a steel plate is considered laminated, the best calking should be done with bevel edges; planing is preferable to shearing, which is liable to tear the fibers. At least one large manufacturer prefers square edges for calking. Split calking from a square-edged plate is dangerous as a split may reach some of the rivet holes.

Diameter of Pipe. With alternate inside and outside courses, diameter is measured to inside course. Allow $\frac{3}{16}$ in. in computed diameter for fitting outer course over inner; rivets will take it up when joints are made.

Size of Pipe. Riveted steel pipes 30 in. in diam. are small for good workmanship, although they have been riveted in field; 36-in. pipe is more easily riveted, and is ordinary minimum. Pipes have been made up to 18 ft. diam.

LOCK-BAR PIPE

Lock-bar pipe is in use on 30-in. line, Coolgardie, Australia; on 48-in. line, Portland, Ore., and in 66-in. Catskill conduits, Brooklyn, in Seattle and many other pipe lines. Pressure-thickness-diameter diagram is given in Fig. 164. Specification for lock-bars at Seattle: "Shall be a steel that experience has shown to be adapted to work proposed; shall be equal in quality to steel specified for rivets of riveted steel pipe, and shall be subjected to same tests. It shall stand cold rolling and working."

Fabrication. Longitudinal seams are formed as shown; circular seams are made as in riveted pipes; joints may be figured for 90 per cent. efficiency (tests by Unwin showed 100 per cent.). Longitudinal edges of plate of lock-bar pipe should be upset to required thickness and after the plates have been rolled to true circles, the lock-bars should be closed down over the upset edges by a hydraulic jack exerting a pressure of not less than 500 tons. Lock-bar pipes accepted in 1911 for 66-in. pipes in Brooklyn had plates $\frac{7}{16}$ in. thick. These bars were of "extra dead soft" steel, composition: carbon not over 0.1 per cent., sulphur not over 0.05, phosphorus not over 0.04, copper 0.18 to 0.25. This steel proved satisfactory; the pipe maker said it was best metal yet used in lock-bars. Later the steel mills and pipe makers contended that the copper had been of questionable benefit if not objectionable. Careful investigation showed no advantage in annealing the bars after rolling, if properly handled on the "hot beds" at the rolling mill. Annealing would add a few dollars per ton to the price of bars. After a bar has been locked the space between the upset edge of the plate and the bottom of the groove in the bar should not exceed $\frac{3}{16}$ in.; if greater, or if bar shows any sign of cracking, it should be cut out and a new bar substituted.

Failure. Under 166-lb. test pressure, a 52-in. lock-bar steel pipe, $\frac{5}{16}$ in.

thick, burst; stress in steel was about 8800 lbs. per sq. in. Lock-bar broke transversely in 11 places, 8 to 15 in. apart, and cracked longitudinally. Metal showed crystallization. Two or 3 min. after break, pipe collapsed at 3 points upstream, first 1700 ft. away. Here 9 pipes, each 30 ft. long, took

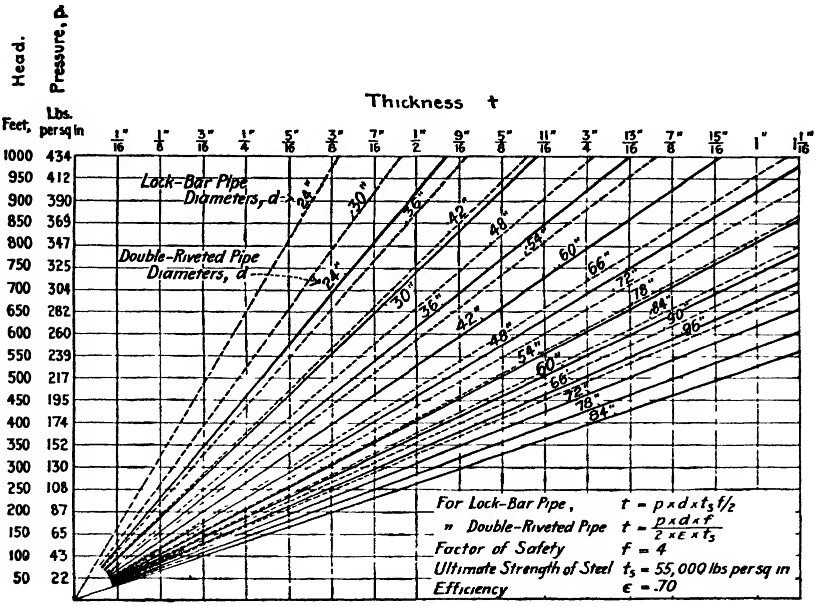


FIG 164.—Pressure-thickness-diameter diagram for lock-bar and double-riveted steel pipe.
(Catalog of East Jersey Pipe Corporation, N Y. City)

this shape: Pipe was not backfilled; two concrete anchors on either side prevented collapse from spreading. At two points about 1000 ft. farther upstream, 9-pipe and 4-pipe lengths, respectively, took this shape at places not backfilled. There was solid backfilling between breaks. These breaks, due to faulty metal in one bar, are not an argument against lock-bar pipe. Of tens of thousands of bars made in U. S., it is stated, breaks could be counted on one's fingers. (E. N., July 27, 1911.)

Lynchburg, Va., 30-in. lock-bar steel pipe, pressures 83 to 175 lbs., $\frac{1}{4}$ and $\frac{3}{8}$ in. thick. Curves were made by special castings; 30-ft. lengths were joined in the trench by cast-steel sleeves, with poured and calked lead joints. Manholes were provided at convenient points for future interior painting and at bottoms of steep grades for exit in case laborers should slip down the smooth inclines which could not be reclimbed. Pipe is anchored to concrete pedestals on steep grades. At small stream crossings undermining of the pipe is prevented by small concrete walls parallel to the pipe and downstream from it. (E. R., Sept. 1, 1906, p. 228.)

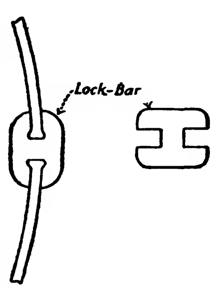


FIG. 165.—Lock-bar pipe. Longitudinal seams.

STEEL PIPE DETAILS

Plugs and Manholes. For passing hot rivets into pipe, holes are made in plates next to field joints, 1¼ in. diam. for ¾-in. rivets, to 2½ in. for 1¼-in. rivets; these holes are ultimately tapped and closed with steam-fitters' screw plugs with tight-fitting threads; the plugs should not project far into the pipe. Wrought-iron screw plugs used in rivet-passing holes have given no trouble, beyond a little leakage easily repaired by calking; cast-iron plugs are poor practice. Plugs of form shown in Fig. 166 were used on Catskill aqueduct pipes. Because of slight yield in compression when screwed in and action of water pressure when in service, they are tighter. Threads must fit tightly. Soft lead grommets or washers are sometimes used under shoulders of plugs and squeezed tightly against pipe plate. Openings in $\frac{7}{16}$- and ½-in. plates should be reinforced by plates riveted

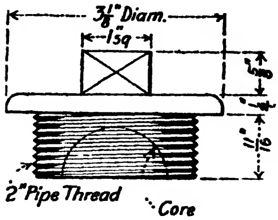


FIG. 166.

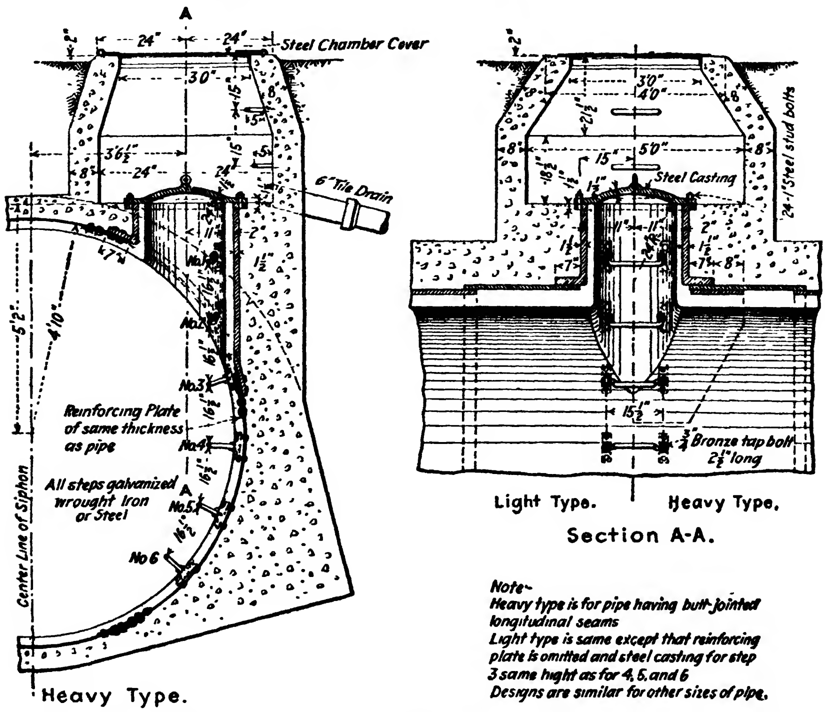


FIG. 167.—Manhole connections, steel pipe siphons, Catskill aqueduct.*

on, or in other approved manner, and closed by threaded plugs, so as to be water-tight. At a manhole put a thin plate between the casting and the pipe, for calking. Manholes on Catskill aqueduct steel pipe siphons are cast steel; specials are connected by cast- or forged-steel flanges.

* For sizes of pipes common in waterworks, boiler types of manholes are usual, spaced 1000 ft. or more.

Steel Pipe as Bridge. A pipe subject to bursting pressure and used at the same time as a beam or bridge should be figured so that the maximum stress resulting from the two conditions will not exceed the allowable stress. If, comparing the total weight of the beam (pipe and contained water) with the safe strength, the thickness is not sufficient for beam strength, rather than increase thickness, calculate body of pipe for bursting pressure only, and rely on longitudinal members (angles, tees, etc.) riveted to top and bottom for beam action. In determining thickness, allow liberally for rusting and deterioration in such exposed places as pipe bridges are usually built in. Two 8-ft. steel pipes, supported on masonry piers, were used for stream control at Olive Bridge dam for 2 yrs. without cover. There was one 42-ft. span with full pipe, one 47-ft. span half full; when removing the pipes, one span of 75 ft. occurred for 1 day with pipe $\frac{1}{4}$ full; no deflections or deformations in the pipes were observed. These pipes had two double-riveted longitudinal seams, single-riveted ring seams 7.5 ft. apart, and plates $\frac{7}{16}$ in. thick.

PICKLING AND COATING STEEL PIPE

Pickling. Pickling under Catskill aqueduct specification, by East Jersey Pipe Co., was done as follows: Each plate was bent in rolls, which thoroughly broke the scale on the inner side but had less effect outside. Acid and washing tanks of wood, in the same shed with the heating ovens, were set in the ground nearly flush with the floor. Wash water was kept alkaline by occasional addition of soda ash. Acid solution was kept at strength of 5 per cent. of oil of vitriol (latter practically 93 per cent. pure sulphuric acid); it was heated to approximately 125° F. and agitated by steam jet discharging below the surface at one corner. Steel was in the acid tank about 15 min., then immediately lowered into the wash tank, for only 2 or 3 min. After removal from the second tank, pipes, if to be coated, were almost immediately run into the oven and heated for dipping. Pipes, after pickling, were free from scale, and of a uniform steel-gray color. Heat of the acid bath is sufficient to cause the pipes to dry rapidly when lifted into the air from the wash tank. Tests showed that the breaking of the scale by bending or other fabrication helped materially in removing it; $3\frac{1}{2}$ per cent. acid solution was quite as effective as 5. Scale is insoluble in acid, and is removed by the action of the acid on the metal beneath. Weak acid solutions attack cast iron vigorously; a little heat, about 125°, greatly accelerates this action; therefore such fittings should not be attached until after the steel has been pickled, if this be practicable, or cast-steel fittings should be used.

For pickling steel plates, the Brooklyn Navy Yard uses a muriatic acid and water bath, about 10 per cent. acid. Plates, on edge and separated about 1 in., remain in the pickle 24 hrs.; then they are removed and washed in a solution of lime in water for about 20 min.; they are not washed between the acid and alkali tanks. After final washing, plates are stored in a shed without further treatment. No precautions are taken to prevent rusting; this is looked upon as desirable to make sure that all mill-scale is removed. Before the completion of a ship, the plates are wire-brushed to get the loose rust off and then painted. Apparently about half the total rust can be brushed off.

On some of the Catskill aqueduct work, after pickling, whitewash was applied to protect from rust between fabrication and the application of the mortar lining. Heavy lime whitewash was made as follows: To 1 bbl. (about 50 gals.) of whitewash there were added 20 lbs. of glue first dissolved in water. Later it was found advisable to add 1 lb. Portland cement to each gal. of whitewash. Brushes proved more satisfactory than spraying machines operated by compressed air. This coating did not prevent light rust, as it did not stay on well enough.

Cleaning and Oiling. In some shops, plates, as received from the rolling mill, are cleaned with dilute acid and brushes before any shop work is done, and cleaned again after shop work and given a coat of boiled linseed oil before exposure to the weather; all painting is then done in the field, preferably with graphite paint. Advantages of oil are that all shop and erection marks made on the bare steel are visible; also any imperfections in the plate which may have passed preliminary inspection may be detected during erection.

Catskill aqueduct specifications for concrete jacket and mortar lining had the following stipulations as to preliminary cleaning. During fabrication into pipe, all plates, at each joint where one plate covers another, shall be entirely free from mill-scale, from any but light rust and all dirt, grease, or other foreign matter. Furthermore, when concrete is placed around, or mortar lining placed within, any section of pipe, surface of plate shall be similarly clean. Degree of cleanliness and freedom from scale required shall be such as produced by thorough sand-blasting or thorough pickling, *i.e.*, such that surface of steel itself is exposed. Initial cleaning may be done at the mill, provided the steel is adequately protected and necessary field cleaning is done.

Applying Coating. Usual method of coating steel pipe is dipping in a hot bath of refined asphalt or compound containing asphalt, immediately after cleaning, at the shop. A perfect, homogeneous coat would be a non-conductor; coats generally contain minute air or gas bubbles, and during handling and transportation abrasion occurs; also the coating itself may not be impermeable under high pressure; thus water finds direct contact with the metal and electrolysis starts. No coating has proved permanently successful; in a short time elasticity and adhesion are lost. In Little Falls, N. J., a 66-in. steel filter influent pipe had $\frac{1}{4}$-in. mortar lining (1 Portland cement: 2 sand) plastered over several coats of neat Portland cement grout on an asphalt coating, and after 5 years' service exhibited no cracks nor tendency to loosen, and no rust had appeared, neither was the elasticity of the coating lost, although sections of the pipe not so treated had no longer any elasticity in the asphalt coating and rusting had extended right up to the mortar. Outside of pipe should have a concrete shell. If this is too expensive, a coating of slaked lime has been suggested by Dr. A. S. Cushman as beneficial.

There is no shop equipped to dip pipe over 8 ft. diam. Dipping vertically in a tank is usually much more effective than painting with brushes. Before dipping, pipes are brought to about 300°, by hot blast or in ovens. Pipes should be revolved in the dipping tank, and great care exercised to avoid foaming, especially while the pipe is being removed; all coating material should be strained through 20-mesh sieve before entering the tank, according to

Culmer, of "Sarco." Gas water and illuminating gas are destructive to asphalt. ("Sarco" is a proprietary bituminous coating.)

Some authorities advise avoiding air currents while withdrawing the pipe from the bath; 30-in. Coolgardie lock-bar pipe was placed in a horizontal position and revolved while hardening to give a uniform coating. Current of air blown through it accelerated the cooling.

Angus Smith Tar Coating. Dr. Angus Smith, in England, about 1840, for protecting iron water pipes, used a coating of coal-tar, distilled to the consistence of melted wax, and an unknown per cent. of raw linseed oil, probably about 5 or 6. Pipe after 35 years was found in perfect condition externally, and to contain few tubercles or deposits inside.

Comparison of Coatings. From an examination of various steel pipe coatings in eastern U. S., it is concluded that a mixture of coal-tar pitch and asphalt has most efficiently prevented or retarded rusting of the interior. Coal-tar pitch is distillate of coal-tar at such a point that naphtha has been entirely removed; the material is then deodorized and reduced to consistence of wax by addition of at least 1 per cent. heavy linseed oil. Ordinary commercial roofing pitch might be suitable after the proper proportions were determined. Some pipe makers state that with modern methods of producing coal gas and its by-products, coal-tar suitable for coating cannot be had, and that coatings made with available tar do not adhere well; asphalt dips are preferred. "Coal-tar pitch heated to usual coating temperature, 300° to 400° F., is very thin; sometimes used straight, but is more or less brittle and subject to chipping; "flowing" point is low, 100° to 145° F. Pitches from residues of petroleum distillation vary, Hydrolene being quite soft when applied and yet somewhat brittle and lacking in adhesion to steel (flowing point, 200° F.); Bitose is very soft and full of oil (flowing point, 206° F.); Texaco is harder and tougher (flowing point, 260° F.). Gilsonite, a mineral pitch, is fluxed with some petroleum product having an asphalt base; Sarco is hard, tough and rubbery, flowing point, 260° F. Pioneer has much the same consistence, but a lower flowing point, 245° F. Nubian products are mineral pitch from carbonization of animal fats collected in an earth pocket; corn oil is said to be the flux; hard, tendency to brittleness; flowing points, 230° to 270° F. All are thicker than coal-tar pitch when heated to coating temperatures, and incline to heavier coatings, which do not adhere so closely to the steel.

Tests. "Comparative tendency to soften and flow is shown by placing a pill of each pitch weighing about 0.2 gram near the top of a glass plate inclined 45° in the oven, heated to any desired temperature for 24 hrs. Softening and flowing points of the harder varieties are determined upon pieces about 1 mm. diam., placed upon thin microscopic cover glasses laid upon the surface of a mercury bath, the bath and dish being covered with an inverted funnel, through the stem of which passes a thermometer. Temperature is raised 2° to 3° F. per min., particles are examined with magnifying lens and temperature at which the edges begin to round over is regarded as "softening" point; temperature at which the pitch begins to spread where in contact with a glass is called "flowing" point. For coal-tar and soft pitches, 1 gram is placed on a washer having an opening 8 mm. diam , suspended in a 400 c.c. beaker of distilled water about 7 cm. from the bottom; temperature is raised 2° F. per min. and point at which pitch runs through washer and touches bottom of beaker is flowing point.

"For determining tenacity with which coating will adhere to steel: Sample is brought to coating temperature and into it are dipped, part way, clean steel bars about 8 in. long previously heated to same temperature; after about 3 min., the

bars are removed, allowed to drain and cool. One set is chilled to 32° F. and another to 0° F., ice and salt being used if convenient; then strike as heavy blow as possible over the edge of an anvil, repeating once or twice on each side, the blow being delivered above the coating. If latter does not chip, blows are repeated upon coated section. Firm, tenacious coating will not chip off at either temperature, except at the point where the blow is delivered; a fair coating may hold at freezing and chip slightly at zero, while a poor coating will be likely to come off in large scales at either temperature.

"Elasticity may be determined by strips of tin plate 2 in. wide heated to coating temperature and dipped in hot pitch. After setting, the strips should be chilled to same temperatures as the steel bars, and bent quickly 45°. Tough, elastic pitch should not crack.

Springfield Pipe. "Coal-tar pitch of standard quality mixed with 3 per cent. raw linseed oil was adopted for 60,000 ft. of 42-in. lock-bar pipe, Springfield, Mass., by Hazen & Whipple; applied to 30-ft. lengths by East Jersey Pipe Co., Paterson, N. J. Pitch was run from barrels into a small tank, where it received the linseed oil; it was then drawn into a vertical dipping tank, and kept constantly at 350° F. by steam coils. Pipes were first cleaned of mill-scale by wire brushes, a most important step, as rough adhering particles cause spreading of pitch upon draining, resulting in uneven thickness over an area surrounding each particle. Each pipe was then placed on rollers in front of a blast and heated until the end farthest from the blast was sizzling hot; the end nearest the blast was put in the pitch first, compensating somewhat for the slightly lower temperature at the bottom of the tank; top and bottom temperatures were kept within 10° or 12° of each other. Excess coating was drained into the tank and the pipe then stood upright on an iron plate until the coating was thoroughly set and nearly cold. Five pipes were put on each flat car, blocked up free of the floor and protected one from another by canvas strips. With one tank and two hoists, about 1500 ft., or 10 carloads, could be coated in 1 day. Freedom from troubles during coating was remarkable. Constant loss of volatile compounds was compensated by adding tar oil. Fresh pitch and linseed oil were added as the bath was depleted. At one time after the addition of fresh pitch, frothing occurred; bubbles so formed inclined to stick to the pipe and cause uneven coating; probably due to moisture in the pitch; it was overcome by the addition of a little linseed oil."*

Specifications for Coating. "After testing to satisfaction of the engineer, pipe shall be thoroughly dried immediately, then cleaned, and heated in a suitable oven to 300° F., after which it is to be thoroughly coated by being dipped vertically in bath of ———— asphalt, manufactured by————————, or such other material, equally good, as the engineer may designate. This coating shall be smooth and hard, yet tough, elastic, durable, strongly adhesive to the metal, and shall be free from blisters and bubbles. Bath shall be heated in such manner as to insure constant and even temperature of 300° F., preferably by steam. Dip shall be kept free from sand, grit or other foreign matter. Contractor shall, as often as necessary in opinion of the engineer, empty the tanks and refill with clean material." (Metropolitan Waterworks, Boston.)

"Immediately after being cleaned, and before any discoloration due to rusting has begun, pipe shall be carefully inspected and, on approval, shall be coated by dipping vertically in "Pioneer Mineral Rubber Pipe Coating" or "Sarco Mineral Rubber Pipe Coating," at 400° F., pipe having been heated to same temperature.

* From "Pipe Coating", by G. C. Whipple, before Chemists' Club, Polytechnic Institute, Brooklyn, 1909.

Pipe should be raised from this bath just sufficiently fast to allow coating to solidify evenly over surface of pipe." (Seattle, Wash.)

Thickness of Coating. If the coating is less than $\frac{1}{32}$ in. thick, the pipe should be re-dipped quickly so as to have not less than $\frac{1}{32}$; $\frac{3}{64}$ in., in the opinion of the manufacturers of "mineral rubber," is the best thickness. Pipe makers use different quantities of coating per sq. ft. of pipe; inquiries from five show 0.1, 0.2, 0.23, 0.5 and 0.7 lb. Ton of asphaltic coating contains about 225 gals., 1 gal. = 51.2 sq. ft. covered $\frac{1}{32}$ in. thick, or 34 sq. ft. $\frac{3}{64}$ in.; 1 ton = 11,520 sq. ft. $\frac{1}{32}$ in. thick, or 7680 sq. ft. $\frac{3}{64}$ in. At $75 per ton, 1 sq. ft. costs $0.0065 if $\frac{1}{32}$ in. thick, or $0.0098 if $\frac{3}{64}$ in. Allow for waste.

Coating Outside of Pipe only. Asphalt or a similar dip coating may be confined to the outside of the pipe by whitewashing the interior and then scraping off the coating after the pipe has been dipped. This is the method used in keeping machined flanges free from coating, and would cost at least twice as much as coating both inside and outside. To close the ends of the pipe with bulkheads and load it so as to sink into the dip is less practicable.

Repairs to Coating. If necessary to patch scarred places, one method is to heat the coating by a plumber's blow-torch until the old coating all around is fluid; when the defect is small, the old coating can be made to flow over it, but if large, some coating material previously heated is applied with a brush and the flame turned on, making the new join the old. Coating will take fire if the torch is not used carefully; hard to extinguish, especially in summer.

Coolgardie 30-in. lock-bar pipe was heated to 300° F. and immersed in a mixture of Trinidad asphalt and tar, maintained at boiling point (14 per cent. of 350-mi. length was coated otherwise). Before the coating was entirely hard, fine sand was distributed over the outside and slightly pressed in by rollers; this was expected to retard flowing under high sun temperatures. Australian temperatures range from below freezing to 170° F.* Chief Engineer Reynoldson thinks sanding not desirable. Soft coatings, with a tendency to flow, are not as reliable as hard coatings. Hard coatings may peel from brittleness in cold weather, but are denser, more durable, and in closer contact with the metal.†

Burlap covering saturated with hot asphalt compound has been spirally wound on to lockbar and riveted steel pipes by East Jersey Pipe Co., Paterson, N. J., at moderate cost. Small seamless pipes wound spirally with strips of the burlap saturated with asphalt can be obtained in the market.

Catskill aqueduct specifications for 66-in. steel pipe are: After pipe has been dipped in mineral rubber coating (as provided in Item —) and just as soon as coating has sufficiently set to prevent flow, pipe shall be wrapped with 10-oz. Calcutta burlap, or equal, in the following or an equally satisfactory manner. Each pipe shall be placed on centers of a wrapping machine where it shall be rotated. The burlap shall be cut into strips 18 in. wide and carried on a reel on a carriage which travels along the pipe while it is rotated. Burlap shall be drawn from the reel by revolving pipe, through a tank of heated mineral rubber pipe coating and then wound spirally on the pipe, except at each end where one turn of each layer shall be wound square to finish off. Travel of carriage for single-coat work shall be about 17 in. for each revolution of

* Black pipe in sun. † E. N., Aug. 24, 1911, p. 221.

pipe, or so that the burlap will be lapped on itself about 1 in. For 2-coat work, the process may be repeated immediately or travel of carriage reduced to 8½ in. The wrapping shall be kept back far enough from ends of pipe to leave rivet holes accessible and not interfere with making of field joints. Tension of burlap for winding shall be sufficient to cause the burlap to lie close and snug on pipe, but not enough to strain or tear it. End to end connections of strips of burlap shall be made by sewing with strong twine. After pipe is laid, riveted, calked and tested, field joints shall be wrapped with burlap soaked in field coating, using one layer or two layers, according as work is done under Item — or Item —. Only 4,600 ft. of a total, 29,000 ft. were so covered.

Coating at Field Joints.—It is not necessary to remove coating, except where it interferes with putting pipes together; it is melted by hot rivets and helps to make joint tight; it may be claimed that coating prevents sheets coming into actual contact and therefore weakens joints by acting as lubricant, but trouble on this account is not experienced in practice.

Defects in Coating. Observations show that, at first, coatings usually adhere on nearly the entire surface. Repeated examinations of pipes in service have shown that most, if not all, bituminous coatings deteriorate greatly in a relatively few years. Asphalt coatings seem to lose elasticity and become brittle. After 2 years' service, coatings, particularly those that go on heavily and lack tenacity, form blisters, in size from a pinhead to a hen's egg, holding alkaline water which contains more solids in solution than the water passing through the pipe, but the iron or steel beneath will generally be bright, and in the vicinity will be found tubercles. Sometimes pittings occur under the tubercles; after 10 years' service, have been found 0.093 in. deep. Danger is that a blistered coating will eventually expose a large surface of pipe. Blisters can be produced quickly by passing a current of low voltage through sea water, using strips of sheet iron for positive electrodes and similar coated strips for negative electrodes, with edges protected by paraffin; 12 to 24 hrs. will produce blisters with some coatings. Thickness of the walls of large blisters is about $\frac{1}{50}$ in.; they are easily broken. Blisters generally are largest on the lower quarter of a pipe and decrease in size and number toward the top. Experience in eastern U. S. has been that steel pipes coated with any asphalt compound show a decided reduction in carrying capacity after a few years. This loss results from: (1) Growth of vegetable matter; (2) deposit of mineral matter; (3) formation of tubercles of rust; (4) formation of blisters in coating. As to (3) and (4) experience on some parts of the Pacific coast has been different, pipes of considerable age having developed neither.

Examination of a large number of preparations, applying tests, showed that asphalt preparations and petroleum products do not adhere closely to steel; petroleum products are more resistant to the action of water, but no more so than coal-tar, which has the advantage of being tenacious with less tendency to blister, especially when used with about 3 per cent. of linseed oil.*

Bitumastic enamel, sold and applied only by Am. Bitumastic Enamels Co., Phila. and N. Y. (Hill, Hubbell & Co., San Fran.), has been extensively used since about 1890 for naval and merchant vessels, pontoon docks, oil-pipe lines

* At Lawrence, Mass., are riveted wrought-iron pipes, some coated with coal-tar pitch, and some with red lead, in good condition after 40 years.

and similar structures in Europe and America, and is reported by users to be quite satisfactory under very trying conditions. In one pontoon dock, the steel plates had been perfectly protected for 20 yrs., and the coating was still in good condition. Although originally developed for marine use, it has been recently specified for steel, cast-iron and wrought-iron pipes. Applied by a brush, it makes a heavy, glossy, black coat. Bitumastic "solution" is used as the priming for the enamel, and in some cases two or more coats of solution are used without the enamel. Solution is less in first cost but not so enduring.

Spring Valley Water Co., San Francisco, Cal.; directions for asphaltum coating: Santa Barbara asphaltum free from impurities broken into 2- or 3-in. pieces. Place in a melting trough and fill the interstices with coal-tar free from oily substances. Boil slowly 4 to 7 hrs. stirring frequently and allowing the refuse to collect in the bottom of the trough. Proper temperature to insure toughness and tenacity of dip is 300° to 305° F. After boiling, the refined asphaltum is poured into another (the dipping) trough, and the refuse is taken from the boiling trough. During the process of dipping pipes, the dip is kept at the same temperature. Consistence is maintained by addition of refined asphaltum from the melting trough, and by the occasional addition of coal tar, to prevent the consistence in the dipping trough from becoming too stiff. Pipes should be given 2 dips, the first lasting about 2 min. Pipe is taken out and drained into dipping trough. After pipe is thoroughly drained, it is again immersed for 3 to 5 min., hoisted out, and drained. In a long run of mixing, the proportion of tar to asphaltum was found to be $\frac{3}{4}$ bbl. (50 gal.) of coal-tar (weighing about 420 lb.) to a short ton of asphaltum. In a case in mind, the melting trough contained a little over 50 tons, and in the preparation of the dip (tempering) 44 bbl. of coal-tar were used. This shows about 1 bbl. coal-tar to one ton asphaltum. Amount of material necessary to purchase will exceed material used by 10 to 15 per cent. Material required per sq. ft. of surface inside and outside (for 2 coats) runs from 0.8 lb. for small pipes, of thin iron, to 1.15 lbs. for large pipes, of thick iron.

MORTAR LINING AND CONCRETE JACKET

Mortar Lining Experiments made for the Catskill aqueduct showed that a lining $1\frac{1}{4}$ in. to $2\frac{3}{4}$ in. thick of mortar or mortar and tile is of sufficient strength to maintain its position within a 9-ft. or larger pipe, acting as an arch; in smaller pipes the arch would be correspondingly stronger. It is feasible by several methods—grouting around a cylindrical form or a mandrel preferred—to produce a hard smooth lining in a pipe. Durability of steel protected by concrete or mortar depends not so much on the ability of the lining to exclude all water as to stop circulation in contact with the steel. Tests on mortar-lining a steel pipe, 9 ft. diam., July to Nov., 1908: (*a*) Expanded metal reinforcement, 1:2 Portland cement mortar, $1\frac{1}{2}$ in. thick, applied in 3 coats, metal in first coat. There was difficulty in working the mortar through the mesh. At the top of the pipe the reinforcement sagged, and so in another trial the plaster was applied direct to the pipe, then the reinforcement was placed $\frac{3}{4}$ in. from the pipe and the plastering finished. Under 75 lbs. water pressure in the pipe, ends bulkheaded, the lining cracked along the longitudinal riveted joint and diametrically opposite. Contact surface of the lining was filled with small cavities. Cost of experimental lining, 20 to 22 cts. per sq. ft.

(*b*) Roebling V-rib metal lath: 2.5 meshes per in.; 3 coats, the first applied through wire mesh, 1:2 Portland cement mortar. Same trouble was experienced at top of pipe as with (*a*). After 2 weeks, 57 lbs. pressure was applied. Largest crack occurred over the longitudinal butt joint. Cost about 14 cts. per sq. ft. (*a*) and (*b*) were not a success; it was shown that cut metal or wire mesh, even though placed close to the pipe, was only in the way and did not aid the mortar at the top in sticking.

(*c*) T-bar reinforcement rings, plain mortar lining. (Leg of T's touched shell of pipe.) (*c*1) 1.5 in. mortar lining reinforced by ⅝ in. × ⅝ in. × ⅛ in.

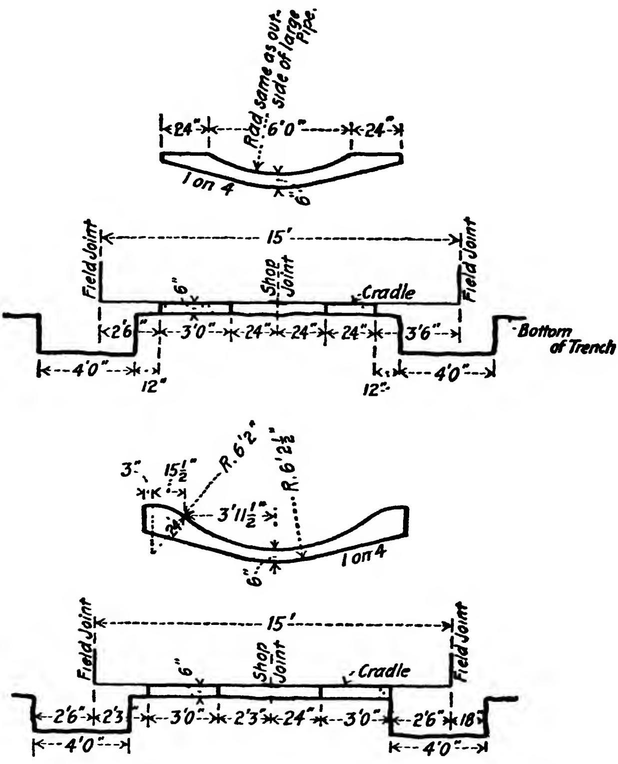


FIG. 168.—9 to 11-ft. steel pipe siphons, concrete cradles, Catskill aqueduct.

T's, spaced 6 to 10 in. centers. (*c*2) 1.5 in. mortar lining reinforced by ¾ in. × ¾ in. × ⅛ in. T's, spaced 8 to 12 in. centers. (*c*3) Plain mortar lining, without reinforcement, 1½ in. thick. A spatter coat of 1 : 2 Portland cement mortar about ¼ in. thick was first applied, the remainder in successive ½-in. layers. Advantage of this method is the probability of the non-existence of voids. After 10 days, 75 lbs. pressure was applied. At 68 lbs. a loud report indicated a break in the lining. Several fine cracks were found, two extending the full length of the pipe. Cost, 16 cts. per sq. ft. for (*c*1) and (*c*2); 9 cts. for (*c*3). The (*c*) tests demonstrated (1) that mortar lining could be built inside large pipes at reasonable cost, and absolutely solid against the shell; (2) that there

is probably no type of reinforcement of appreciable benefit during placing of lining, although some may aid in holding lining in place if cracked in service. (*c*) was more satisfactory than (*a*) or (*b*).

(*d*) Concrete and vitrified clay tiles and grout: (*d*1) Reinforced concrete tiles, laid in mortar and plastered with finish coat; tiles in three sizes, 6 × 8 in., 6 × 12 in., 8 × 12 in., ¾ to ⅞ in. thick; total thickness of lining, 1.5 in. (*d*2) Curved vitrified clay tiles, 8 × 12 in., ⅞ in. thick, with tongue-and-groove longitudinal joints, laid in mortar, and plastered with finish coat; total thickness, 2.5 in. (*d*3) Plain mortar, placed by grouting around mandrel; thickness, 2.5 in. (*d*4) Same as (*d*3) except for addition of woven wire triangular mesh, 2 × 4 in. (*d*3) and (*d*4) placed by grouting machine under 100 lbs. pressure. Under 75 lbs. pressure (*d*) linings developed smaller cracks than did (*a*), (*b*) and (*c*). Tests demonstrated that these linings were practicable and

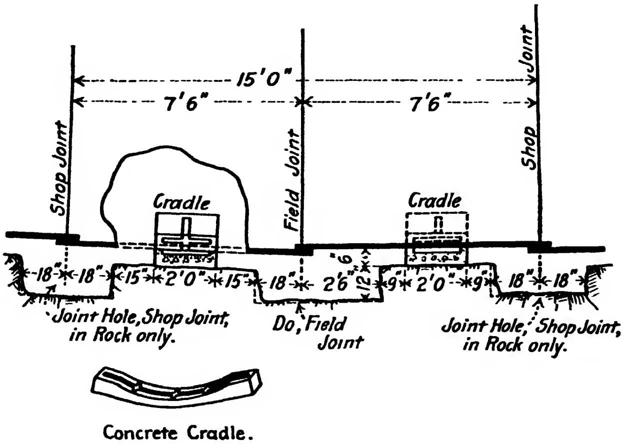


FIG. 169—Keyed cradle and excavation for field joint in rock. (Catskill aqueduct.)

successful work could be depended upon. Reinforcement in (*d*4) seemed no obstruction to grout.

(*e*) Unreinforced plaster and grout: (*e*1) Plain mortar coating, 1.25 in. thick, plastered directly on pipe; (*e*2) plain mortar, 2 in. thick, grouted around mandrel; (*e*3) same as (*e*2), except addition of 2×4 in. triangular mesh reinforcement. After 9 days 51 lbs. pressure was applied. No evidence of rust was found on the lining after 5.5 weeks. Grout was not placed under greater head than that obtained by pouring from the top of the pipe.

In Catskill aqueduct are fourteen steel pipes, 9.5 to 11.25 ft. diam., totaling 33,031 lin. ft.; plate $\frac{7}{16}$ to $\frac{9}{16}$ in.; heads 50 to 340 ft. In view of the knowledge concerning the usual coatings, the engineers decided to clean thoroughly plates of large steel-pipe siphons (see p. 313), cover the pipes, after laying, with a 6-in. jacket of concrete and line with 2 in. of cement mortar. Pipes were tested at the shop and shipped in 15-ft. lengths, laid on concrete cradles in the bottom of the trench (Figs. 168 and 169), tested in trench under normal hydrostatic pressure and made tight, and while still full of water and under

full pressure, jacketed with concrete. These precautions were necessary, as variation in shape would crack the jacket; pressure was maintained until the jacket had hardened; the water was emptied out, the pipe cleaned and the lining put in. The lining in these Catskill aqueduct pipes consisted of 1 part Portland cement and 2 sand, approximately. The first step in lining was to place the invert about 8 ft. wide, screeded to template; cylindrical wood forms in segments were then set up and the remainder of the lining poured as a thin grout or mortar, through holes in the top of the pipe. These holes were tapped for 2-in. pipe in the up-hill end of each 15-ft. length, through which hot rivets were passed, and later mortar poured. Mortar, or grout, was mixed to a thick creamy consistence and allowed to flow into place as uniformly as possible. When the section was filled to the top, pouring of grout continued until grout ran from the air inlet; then headers of steel pipe were screwed into the inlet and outlet and filled with grout so as to put a head of at least 4 ft. on the highest part of the section; the headers were kept filled with grout until the grout had set; the pipes were then removed, and the holes made water-tight by screw-plugs. Some fine cracks appeared in all linings during the first winter, pipes being empty; this was not a cause for apprehension. One siphon was lined with a cement gun (see E. R., July 1, 1911); this lining was built up in successive layers and finished with a trowel to a very smooth surface. After 5 years, linings are in excellent condition.

Adhesion of mortar lining and concrete jacket was not absolute everywhere as was proved by sounding, but the separation was very slight as determined by cutting into selected hollow-sounding places. Some cracks and separations had been predicted and probable results investigated at the laboratory before the contracts were prepared. In one test six steel plates 8 × 16 in., 12 gage, were pickled, then rubbed with emery cloth and placed horizontally in a tank, separated from the bottom of the tank by alberene stone blocks and from each other by wood strips. First pair was without protective covering; second pair had upper surfaces protected by 2½-in. mortar slabs separated from the plates by 0.04-in. metal strips; third pair was protected by 2-in. mortar slabs cast directly on the steel and apparently adhering firmly. Tank was filled with Croton water 4 in. above the top slab and renewed twice monthly. After 2 yrs. the first pair showed heavy corrosion; second, very slight corrosion, most of which washed off; third, part of surface was clean and wet, the remainder dry and covered with firmly adhering particles of mortar, no rust being found when the mortar was broken off. In another test four concrete slabs 15¼ in. diam., 3 in. thick, were made of a rather dry mix, 1 part of cement, 2.7 gneiss screenings, 6.3 crushed gneiss, by weight; 4 soft steel rods, ¾ in. diam., 3 in. apart, were placed in the middle of each slab, so that there were at least 1½ in. of concrete in all directions around each rod. When set, 2 slabs were immersed in water in tanks 2 ft. deep; 2 had bottomless galvanized iron cylinders, 15¼ in. diam., cemented to them and water maintained 20 in. deep. Latter slabs, submitted to percolation, leaked freely at first, but became gradually tighter; during the last few months there was little leakage. Tests were made in open air, July, 1907 to Mar., 1909—20 months. Slabs were broken and all rods found perfectly free from corrosion.

Concrete as broken was found thoroughly saturated, showing that the water had full access to the rods. Experiments briefly mentioned in E. R., May 8, 1909, indicate that painted bars embedded in concrete and subjected to moisture rust more quickly than unpainted ones.

Metropolitan Waterworks,* Boston, lined a steel pipe 80 in. diam. at Newton, Mass., with 2 in. of Portland cement mortar by pouring in forms. Due to thinness of the lining, special jacking had to be done to take the deformation out of the steel pipe before the lining was poured. Inside steel forms were used, adjustable to give 2 in. thickness in deformed portions. No troubles with air pockets were experienced.

LAYING RIVETED PIPE†

Placing Pipe. Calking edges of longitudinal seams should be set face up. Put no permanent blocking under the pipe; block up for riveting and remove the blocks as backfilling progresses. Artificial ventilation for inspection of the inside of the pipe is a necessity under some conditions. After the foreman

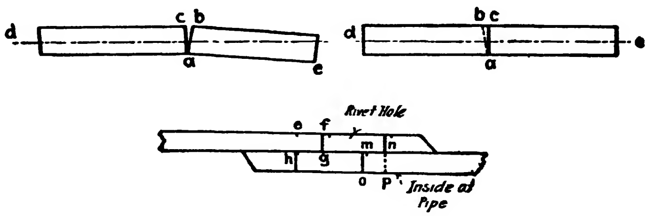


FIG. 170.

says a length of trench is ready for pipe-laying, it should be carefully inspected to see that it is down to grade and that the sides conform to required dimensions; for the latter, a template is useful. Any projections inside the limits should be removed before the pipe is lowered into the trench. While the pipe is on skids over the trench, it should be carefully inspected inside and outside to see that the coating is intact; injuries should be repaired. In entering a pipe into one already laid, great care should be taken to prevent distorting edges of plates. In temporarily bolting a new section to that already laid, the top of the suspended pipe is entered into the other and a bolt inserted through the top rivet hole. Care should be taken to see that longitudinal seams are all spaced approximately equally on each side of the center line. Pipe is then gradually lowered and bolts inserted on each side of its top. To guide the inside plates, pries or bars with sharp, chisel-like ends are used. In case the edges of either plate are distorted, they should be hammered just enough to permit entering. While the pipe is being adjusted to line and grade it should, if practicable, be suspended from the top of the trench by a sling and braced on each side. Should rivet holes come "blind" more than $\frac{1}{32}$ in., remedy this before leaving the pipe. For example, when ready for bolting pipe might lie in position first shown. (Fig. 170.) To bring the pipe to line bolt a few holes on side *a* and then jack end *e*. Thus the rivet holes will come fair at *a* and will pass

* E. R., Oct. 28, 1911, p. 493.
† Applies also to lock-bar pipe, except as to longitudinal seams.

each other at *cb*; when the holes are cut out for the rivets, the plates will cut back from the lap, thus not weakening the joint. When the holes come blind, so that slight drifting will not remedy them, the joint should be fitted before the rivet gang starts on the seam. In Fig. 170 the part *mnop* should be gouged out, reamed or drilled, as determined by the inspector. On no condition, except when rivets are driven from inside of pipe (as in the bottom), should the part *efgh* be cut out, for if it is, the rivet head will not cover the hole. It may become necessary to drill an entire new set of holes; the inner course should be pushed well into the outer course and drilled from inside. All burrs should be removed from holes by a countersink or burr reamer. Circular seams should be well bolted before leaving, using bolts of same diameter as the rivets, spaced not exceeding six holes apart. In no case should drift pins be allowed to remain in holes in place of bolts.

Field Riveting. As many rivets as possible should be driven from outside, using a snap to form the heads. These can be driven to the points where radial lines make an angle of 45° from the bottom with the vertical. Two methods may be employed: either to use a snap, or to make a hammered head, as in boiler riveting. Riveters, especially boiler riveters, prefer a hammered head, as they say it insures greater tightness. This may or may not be true. It is true in case holes come blind and have to be cut out, as then a hammered head entirely covers the hole, whereas a snap head may not. It is easier to make a hammered head than a snap head, as a snap has to be struck with a heavy sledge and room inside the pipe is necessarily limited. All seams driven on the bank should have snap heads, as the joint is accessible all around by rolling the pipe over. Plates should lie in close contact at both calking edges. If not well laid up, sufficient rivets should be cut out, the plates hammered up, and new rivets driven. After a seam is driven, all rivets should be carefully inspected. Any rivets with defective heads should be cut out. All rivets should be tapped with a light hammer, at same time holding a finger against the rivet head. A loose rivet is thereby at once detected. Loose rivets and those that jar *at all* should be *cut out* and new rivets driven. *Do not* allow a loose rivet to be calked. When rivets are cut out, it is likely that rivets close by may be loosened, and if any are found these should be cut out also. Sometimes the head of the last rivet in a longitudinal seam may lap over the edge of the plate. This should not be allowed, as the head will not lie in close contact and a leaky rivet might result; the plate should be chipped before driving the rivet. Or the plate may lap over a rivet head; this is worse, as poor calking would follow. Chip the plate here too, so that it will lie in close contact with the plate in the other course. When holes come so blind that hammered heads will not cover the holes entirely, burrs should be driven into that part of the hole not filled by the shank of the rivet, and a perfectly tight job will result.

Field Calking. Wherever possible, field calking should be done by machine. Before striking with a calking tool, the plates should lie in close contact at the beveled edge. The seam is first gone over with round-nosed fuller tool; after this a half-round fuller tool is used to finish off. To make a fine appearance a cold chisel may then be used to cut off the thin scale of metal made by the

fuller tool. This is not necessary, however, as the seam will be tight after using the half-round tool. Split calking should not be allowed, that is, to split a plate so as to make a beveled edge lie in close contact with the other plate. Neither should strips of sheet metal be driven in between plates to accomplish same result as split calking. The only proper way to remedy such trouble is to cut rivets out, lay the plates up well and drive new rivets. In the trench each field joint may be tested by an apparatus which puts pressure on a single joint, but it has been found that rigid inspection is better than any test. Tap every rivet and try every part of both inside and outside calking with a *very* thin-bladed knife, or a machinist's "feeler."

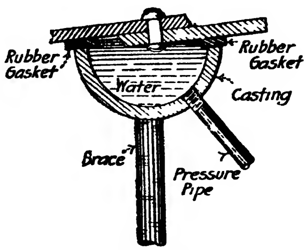
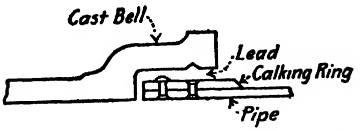


FIG. 171.—Apparatus for testing separate field joints (steel pipes).

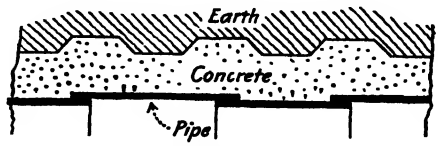
Protection of Coating. Loading on, and unloading from, cars and transporting to the trench should be carefully specified and inspected. In one case unloading from cars was successfully accomplished by means of rope slings and a tall derrick. Two methods were used on one job in transporting from cars to the work; while snow lasted, one pipe at a time was put on a specially constructed low sled, and rolled off sideways at the proper place; when there was no snow, pipes were put on to a movable skid lying on rollers in a light truck; at the right location the skid and pipe were slid endwise from the wagon, and then the pipe rolled sideways from skid; both were successful. After being strung along the trench, two or three pipe sections are often riveted together; in case it becomes necessary to move them longitudinally, a successful method is to roll the pipe sideways on to skids and move the skids and pipe on rollers. Placing the pipe in the trench and connecting it is sure to mar the coating. In cold weather there is no trouble in making everybody who has to walk on or in the pipe wear rubber shoes or boots, but it is impossible to keep them from dropping tools. In warm weather it is impossible to make men wear rubbers or to keep men with hard boots off the coating, though as the coating is more pliable at this time, not so much damage is done. Other sources of damage are riveting gangs, dropping tools, sitting on boards whose edges cut into the coating, dragging kegs of rivets, etc.; also stones in the backfilling. It is specified sometimes to use canvas for covering inside and outside of the pipe to protect the coating during the above operations, but this is impracticable; the pipe often has water in it, and the canvas would freeze stiff in winter; it cannot be laid across joints to be riveted, and often a large quantity would be required to protect all the pipe exposed, besides requiring much labor and inspection to see that the canvas is always spread.

Anchorages and Connections. Steel pipes should be reinforced and anchored with concrete at gate chambers, for 20 to 30 ft. Curves, if of small deflection, need not be anchored. Sides and bottoms of trenches in firm earth may be notched, to save concrete and increase the strength of the anchorage. At chambers or connections with cast-iron pipes, steel pipes should be reinforced with a band against which to calk the lead joint; this band should be riveted

to the pipe with the heads in the row farther from the end of the pipe countersunk on both sides and the end row countersunk on the inside only, the button heads furnishing a grip for the lead. Depth of bells for such connections



Connection of steel pipe to valves or cast-iron pipe.



Anchorage of steel pipe in earth.

FIG. 172.

should be 6 or 7 in., but $5\frac{1}{2}$ in. may suffice for small pipes. Lead joints of bell-and-spigot type should be used for connecting steel pipes to valves, Venturi meters and similar castings wherever contraction of the pipes would otherwise break the castings. For flange connections to specials and valves in a steel

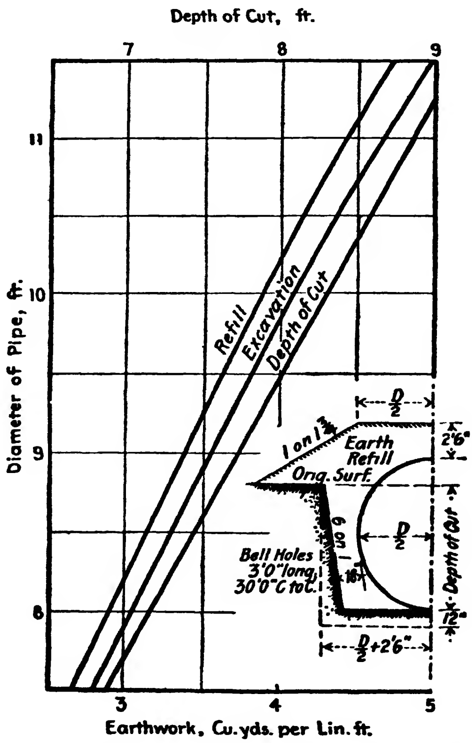


FIG. 173.—Uncovered steel pipe, in compact earth. Earthwork quantities.

pipe-line use rolled, cast or forged steel flanges—not cast iron. Steel pipe should be anchored against temperature movements. Never leave a long length uncovered in winter between two lengths anchored, or the rivets in the

circular seams of the exposed portion will be subjected to shear by temperature changes.

Earthwork. One foot each side of the pipe is sufficient width of trench in earth, whether sheeted or not, for riveting pipe. For bell holes, $1\frac{1}{2}$ ft. around the pipe and 6 ft. long is sufficient; this can be reduced to 1 ft. on sides where sheeting is necessary. Weston aqueduct inverted siphons: Cover of 2 ft., except one stretch, 2.5 ft.; top width, where embanked, 10 ft. minimum;

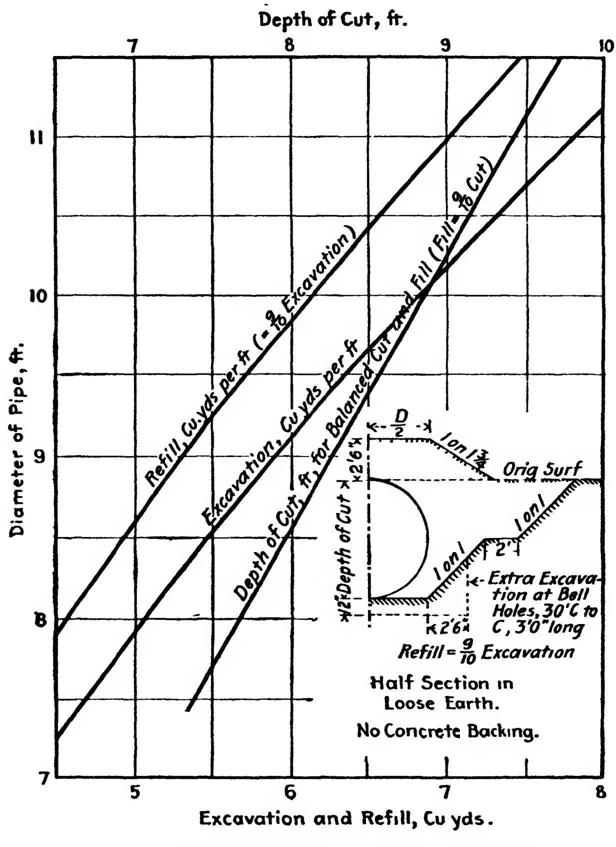


FIG. 174.—Uncovered steel pipe, in loose earth. Earthwork quantities.

slopes, $1\frac{3}{4}$ to 1. Jersey City conduit: Cover, 3.0 ft. Ogden, Utah, Pioneer power plant: Cover, 3 ft. 72 in. Brooklyn pipe: Cover, 3 ft. Catskill aqueduct, 9-ft. to 11-ft. diam. (in country): 3-ft. cover; 66-in. mains (in city streets): 4-ft. cover.

Erection in Place. Many penstocks and some water-supply pipes have been built up, plate by plate, in place. Plates are bent to shape and punched in the shop and shipped nested; this involves two longitudinal seams. Some makers prefer this procedure for diameters above 8 ft., and claim economy for it; 18-ft. penstocks for Ontario Power Co., Niagara Falls, were built in place

from ½-in. plates shipped flat, the average speed being 48 lin. ft. per day (E. R., Oct., 1904, pp. 462, 504). Freight is cheaper for plates than for pipes.

Alinement of Steel Pipes. Radii and spacing of curves should be selected, if practicable, so that full plates will come between angle points. It is advis-

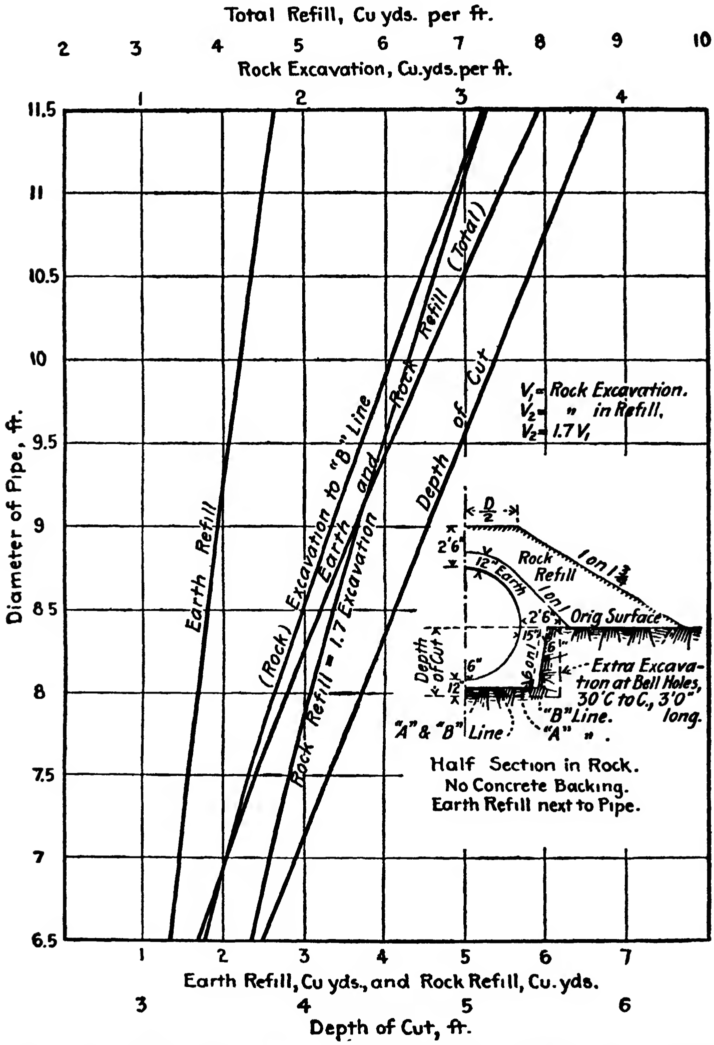


FIG. 175.—Uncovered steel pipe, in earth and rock. Excavation and refill per linear foot.

able to have a uniform bevel, equal on adjacent plates, not exceeding 2° 30′ per plate, or 5° per joint, when shaped into pipe.

To find Radius of Curve in One Plane. With equal angles at all angle points, the center line of the pipe lies entirely outside the theoretical curve, touching it only at the mid-points between angles; also the point of the first angle and

the point of beginning of the theoretical curve are not coincident.* This must be taken into account both in designing and locating curves, ordinary railroad curve formulæ not applying without modification. If l = distance in feet between angle points; T = distance in feet from intersection of tangents to first angle point; I = total angle of deflection; a = angle of deflection at each point; R = radius of theoretical curve; $R = \frac{l}{2} \cot \frac{1}{2} a$; $T = R \tan \frac{1}{2} I - \frac{l}{2} = \frac{l}{2} \cot \frac{1}{2} a \tan \frac{1}{2} I - \frac{l}{2}$.

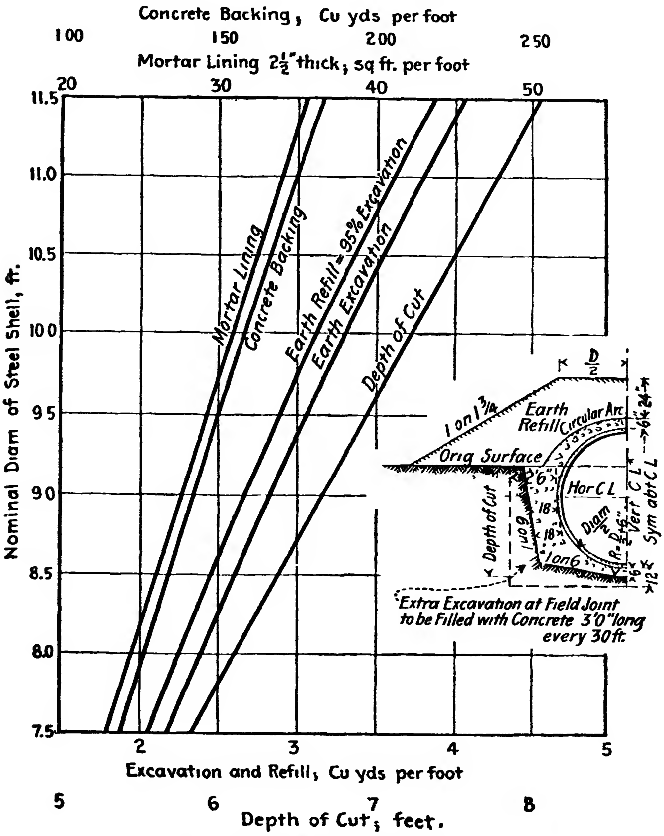


FIG. 176.—Concrete-covered steel pipe, in compact earth. Earthwork, concrete and mortar lining quantities

No particular advantage accrues from having a standard radius. Estimate curves and similar special work at about $\frac{1}{3}$ more per foot than straight pipe, for the cost of making. On long lines it is customary to assume 5 per cent. of the length as curves. Wherever possible, use full-length plates. Beveling both ends of each sheet gives a short radius. Such a short radius makes work a little more difficult and interferes with dipping. Wherever practicable some prefer a radius that will not require more than one bevel on every second sheet; with sheets

* Fig 180, p 337.

7½ ft. long between circular seams this approximates a 20° railroad curve, about 280 ft. radius. If full-size plates are used, curves do not add greatly to the cost; curvature requiring special plates adds to the wastage and thereby to the cost.

The minimum practicable radius would correspond to 2° 30′ off each end of each course, and 12 in. between rivet pitch lines on the shortest side.

Curves in Two Planes. If a detour is required to pass beneath a sewer, or other obstacle, at a given point, use a combination of simple curves. Assume

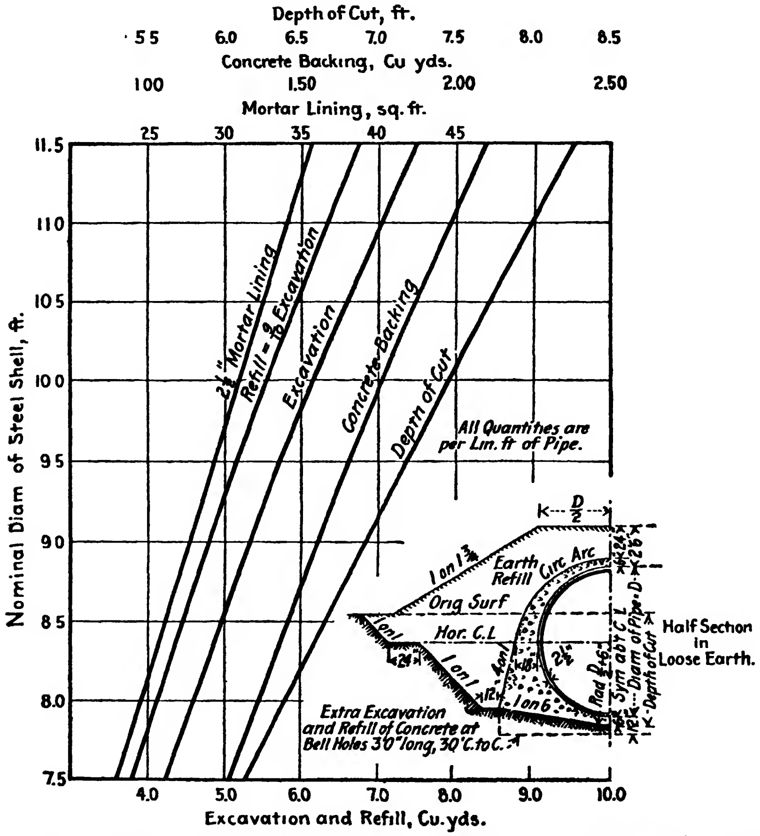


FIG. 177.—Concrete-covered steel pipe, in loose earth. Earthwork, concrete and mortar lining quantities.

point S depressed 7 ft. (See Fig. 181.) Lay curve 1 on the regular grade; twist curve 2 on the tangent of its axis, at A, the point of beginning of curve, so that its end B drops half the vertical distance required; twist curve 3 an equal amount in the opposite direction so as to be tangent to curve 2 at B and have the tangent at S horizontal. Lay 4 like 3, 5 like 2, and 6 like 1. Computation of curves 1 and 6: Total given or assumed angle = 44°; make a = 4° (a is angle between adjoining beveled sheets); first and last angles = ½ a. Assume 7-ft. sheets, 48-in. pipe, radius R = 94.59 ft., using full sheets and no unbeveled

straight sheets between angle points. Make curves 2, 3, 4, 5 of same radius and in same manner, *i.e.*, as chords to the theoretical curve. Fig. 181 gives the elements of one curve. Following assumptions were made: I should be a multiple of 4° 0′; B = 22° 0′ or $2B$ = 44° 0′; R = 94.59 ft.; x = 3.44 ft. A rigid solution is complicated, and so successive approximations may be made. If $I = 24°$, $T = R \tan \frac{1}{2} I$ = 20.11 ft.; $b = \sqrt{T^2 - x^2}$ = 19.81 ft.; $c = T$

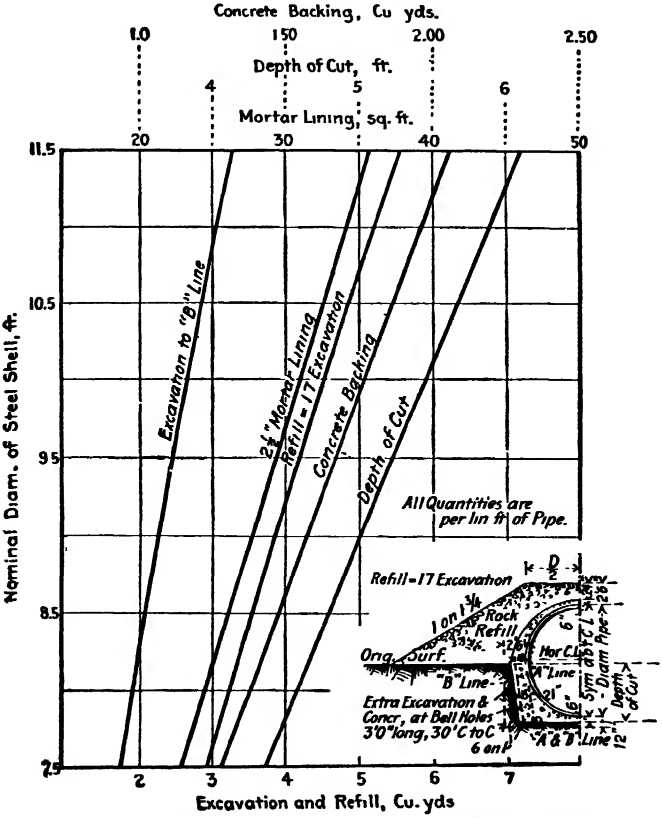


FIG. 178.—Concrete-covered steel pipe, in rock. Rock excavation, refill, concrete and mortar lining quantities.

cos I = 18.37 ft., and $\cos B = \frac{c}{d}$; B = 21° 59′ 41″; $2B$ = 43° 59′ 22″. (See also p. 397.)

SPIRAL RIVETED PIPE

Advantages.—Patents of Abendroth and Stein, of New York, expired several years ago. Pipes of No. 16 galvanized steel have been in use in small water systems since 1904. In San Gabriel, California, pipes under 35 lbs. pressure have given no trouble. Said to be 25 per cent. cheaper than cast iron for sizes under 10 in., and 35 per cent. for 12 to 20 in. Lexington, Mo.,

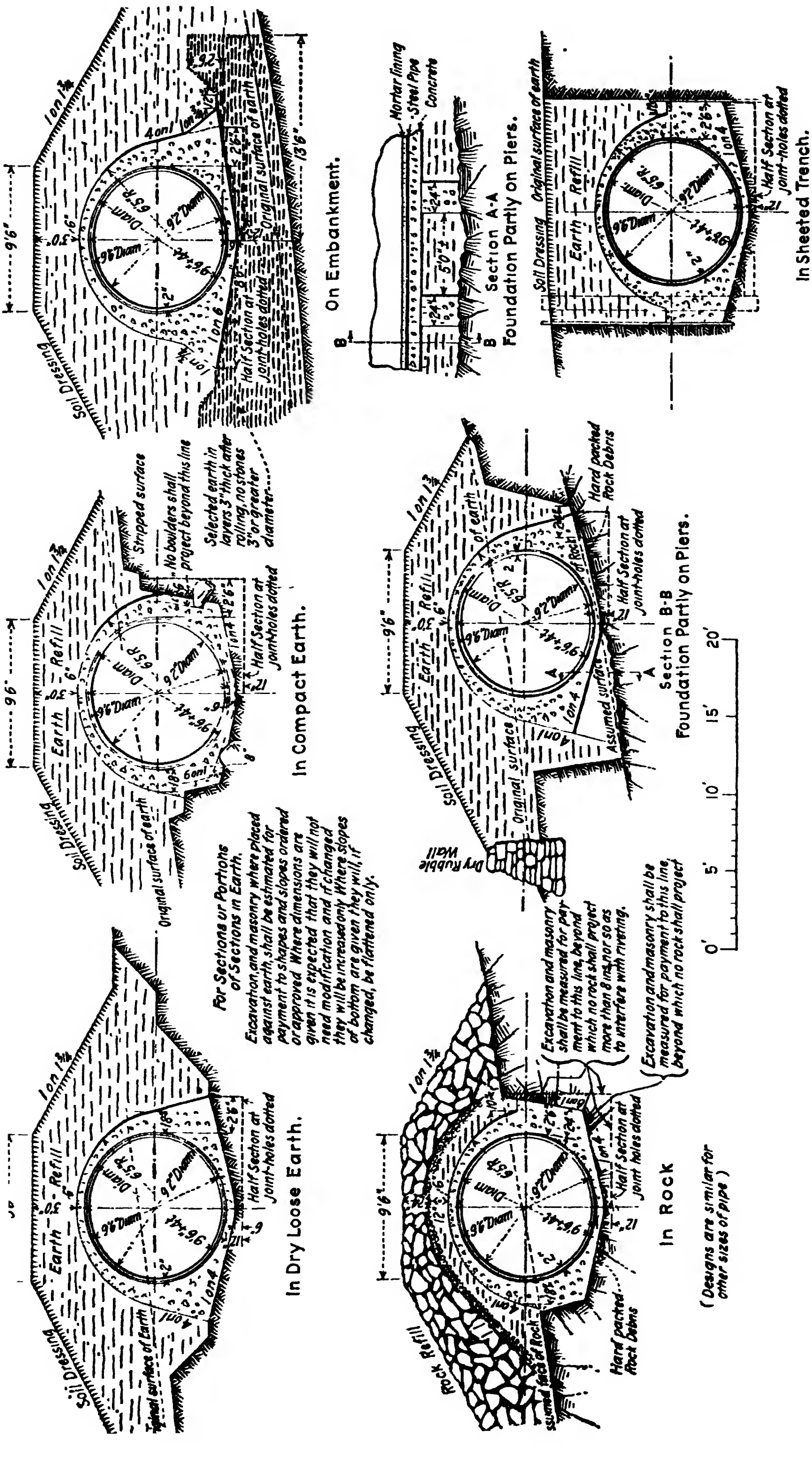


FIG. 179.—Steel pipe siphons, standard types, Catskill aqueduct. Concrete jacket and mortar lining.

pipe was laid in 1887. Spiral pipes at Portsmouth and Gorham, N. H., Brattleboro, Vt., Langerville, Dixfield, Millinocket and Lewisport, Me., were laid about 1902. A 24-in. spiral pipe, No. 12 gage, laid on Long Island in 1887, was uncovered in 1907, apparently in good condition; in addition to tar dip, it was wrapped spirally with tar paper while coating was hot. Pressures in this pipe are light. Spiral riveted pipe possesses great strength for spanning ravines or withstanding heavy earth fills, owing to the wide lap of the helical seam; it also stands greater collapsing pressure than other pipe of same thick-

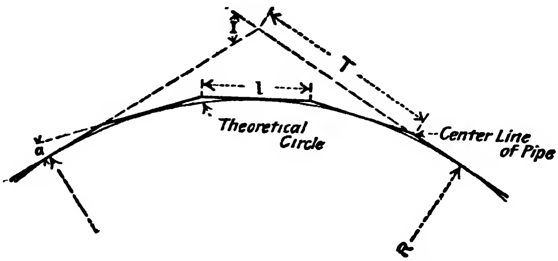


FIG. 180.—Alinement of steel pipes. Radius of curve.

ness. Spiral riveted pipe 6 in. diam., 24 ft. long (No. 12 gage, double extra heavy) has sustained, as beam supported at ends, a total distributed weight of about 2900 lbs.* Each length of pipe, when pressure is specified, is tested to 50 per cent. more than the working pressure before shipping. American Spiral Pipe Works have a testing machine which will test pipe of any diam. up to 84 in. and any length up to 40 ft., with a capacity for 1000 lbs. per sq. in. on pipe of 40-in. diam.

Spiral riveted pipe is used extensively in waterworks and hydro-electric plants for pressure up to 400 lbs., hydraulic mining under heaviest pressure,

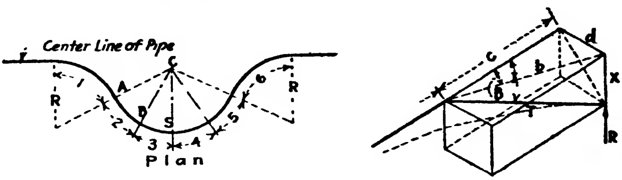


FIG. 181.

paper and pulp mills, suction and discharge on centrifugal pumps, feed-water heaters and for exhaust steam purposes; especially adapted for long lines, where strength, durability and price enter into consideration. It is easily erected. All water pipe is thoroughly protected inside and out by galvanizing or by special mineral rubber composition; the pipe is immersed in a hot fluid bath, and allowed to remain until it reaches same temperature as bath. Many pipes coated in this manner have been in use over 20 yrs. and are still in good condition. Manufactured in sizes from 3 to 40 in. diam., of various thicknesses.

In manufacturing spiral riveted pipe, a strip of sheet steel is wound into

* American Spiral Pipe Works.

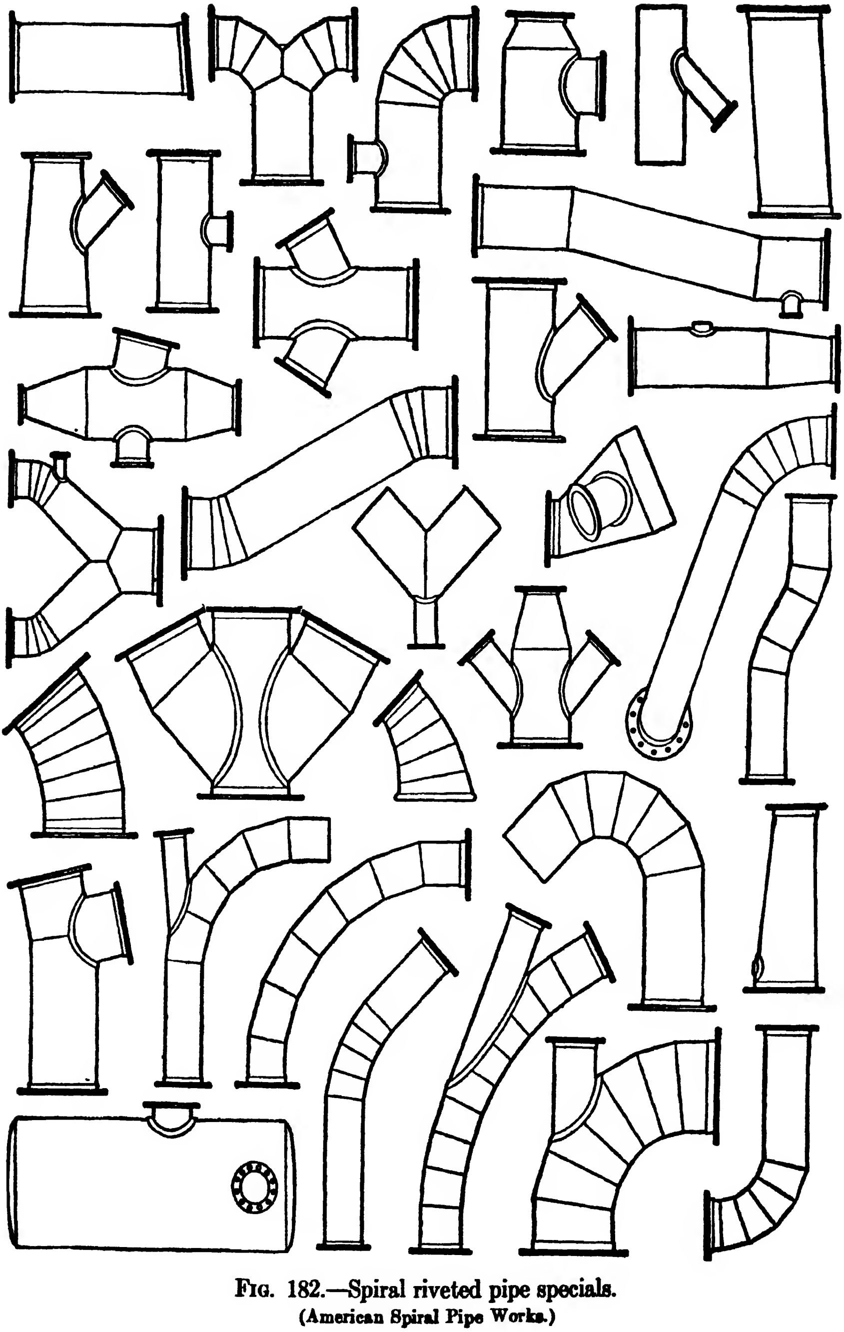


FIG. 182.—Spiral riveted pipe specials.
(American Spiral Pipe Works.)

Table 84. Spiral Riveted Pressure Pipe

(American Spiral Pipe Works)

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
3	20a	1.9	1500
	18b	2.3	2000
4	20	2.4	1125
	18a	3.0	1500
	16b	3.7	1875
5	20	2.9	900
	18a	3.7	1200
	16b	4.5	1500
6	18	4.3	1000
	16a	5.3	1250
	14b	6.6	1560
	12c	9.2	2170
7	18	5.1	860
	16a	6.2	1070
	14b	7.7	1340
	12c	10.7	1860
8	18	5.8	750
	16a	7.1	935
	14b	8.8	1170
	12c	12.3	1640
9	16a	8.0	835
	14b	9.9	1045
	12c	13.9	1460
10	16a	8.8	750
	14b	11.0	935
	12c	15.3	1310
11	16a	9.7	680
	14b	12.0	850
	12c	16.6	1200
12	16a	10.6	625
	14b	13.0	780
	12c	18.2	1080
	10	22.5	1410

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
13	16a	11.4	575
	14b	14.1	720
	12c	19.7	1010
	10	24.5	1295
14	16	12.9	535
	14a	15.9	670
	12b	22.2	940
	10c	27.6	1210
15	14a	17.0	625
	12b	23.7	875
	10c	29.6	1125
16	14a	18.1	585
	12b	25.2	820
	10c	31.5	1050
	8	38.1	1290
	6	44.7	1520
	3	51.6	1880
18	14a	19.9	520
	12b	27.6	730
	10c	34.5	940
	8	41.6	1140
	6	49.0	1360
	3	59.2	1660
20	14a	22.1	470
	12b	30.6	660
	10c	38.3	840
	8	46.2	1030
	6	54.1	1220
	3	65.6	1500
22	14	24.4	425
	12a	33.7	595
	10b	42.2	765
	8c	50.8	940
	6	59.5	1108
	3	72.2	1364
24	14	26.4	390
	12a	36.5	540
	10b	45.7	705
	8c	55.2	820
	6	64.6	1015
	3	78.4	1250

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
26	12a	39.5	505
	10b	49.5	650
	8c	59.8	795
	6	70.0	935
	3	84.9	1154
28	12	42.1	470
	10a	51.7	605
	8b	63.6	735
	6c	76.6	870
	3	90.4	1071
30	12	45.3	435
	10a	56.8	560
	8b	68.7	685
	6c	80.5	810
	3	97.7	1000
32	12	49.1	410
	10a	61.6	525
	8b	74.3	645
	6c	87.1	760
	3	105.8	940
34	12	52.1	380
	10a	65.4	490
	8b	78.8	600
	6c	93.6	715
	3	112.3	880
36	12	55.1	365
	10a	69.1	470
	8b	83.4	570
	6c	97.8	680
	3	118.8	830
40	12	61.1	330
	10a	76.7	420
	8b	92.4	515
	6c	108.5	610
	3	131.8	750

a Standard thickness for spiral riveted pipe
b Extra heavy.
c Double extra heavy.

Working pressure should not be more than 25 per cent. of ultimate strength or bursting pressure: *i.e.*, factor of safety should be 4 when working pressure is practically uniform; for pumping pressures, or where there is probability of water-hammer, larger safety factor is advisable. Galvanized pipe is furnished in any lengths up to 20 ft., and asphalted pipe up to 30 ft.

helical shape with one edge overlapping the other for riveting; the sheet is drawn and formed in such manner that metal to metal contact in spiral seam is obtained, stretching steel on outer lap, slightly offset, in order that pipe may be more nearly smooth on inside. Riveting is done cold by compression, not by hammering, thus insuring complete filling of holes with slight countersink. Pipe comes from machines in a continuous piece, and is cut to any desired length.

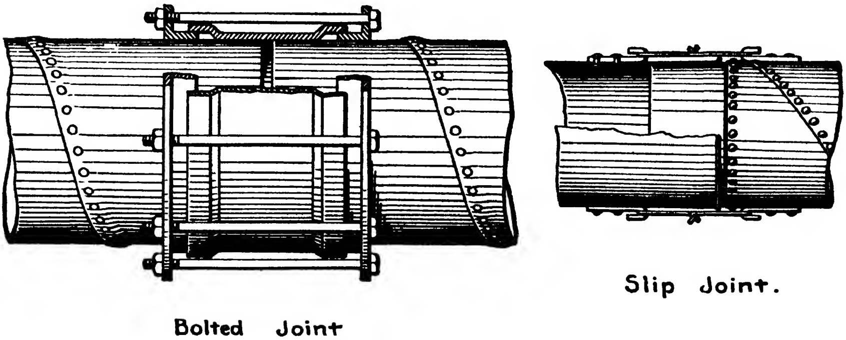


Fig. 183.

Slip joint, see Fig. 183, is used largely for medium and low-pressure work; the sleeve, which is attached to one end of pipe, is wrapped with burlap or canvas soaked in red lead or liquid asphaltum, then driven into the adjoining pipe; the lugs are then connected by wire in order to hold the pipe securely.

Flanges. American Spiral Pipe Works make forged steel flanges attached rigidly to the pipe by use of power riveters, making absolutely tight joints; also make forged steel bolted joints, tight under pressure, which allow slight deflection without leakage, so as to conform to the grade line. (See Fig. 183.)

Lap joints using extra heavy high-hub flanges have met with favor for high-pressure work; extensively used in large power houses; made by heating end of pipe and turning it over the flange, then facing end of pipe; no possible place for leakage, except through gasket; flange is loose and can be turned to any position for alinement of bolts. This joint, with slight modifications, is known under various trade names: Van Stone, Kellog Improved, Atwood, Cranelap, Walmanco, Pittsburgh, Mitchell Recessed Lap, Climax Rolled, and Whitlock Patented Joint.

Shrunk and peened joint (Fig. 185), also satisfactory, is made by boring the flange a little smaller than outside of pipe and then shrinking the flange on while hot; end of pipe is usually peened into flange by hand hammer or expanding machine. Some engineers require large sizes riveted, in addition to peening. There is considerable strain put upon the flange by the shrinking and any other than forged steel flanges are unsafe. This joint is also standard for U. S. Navy.

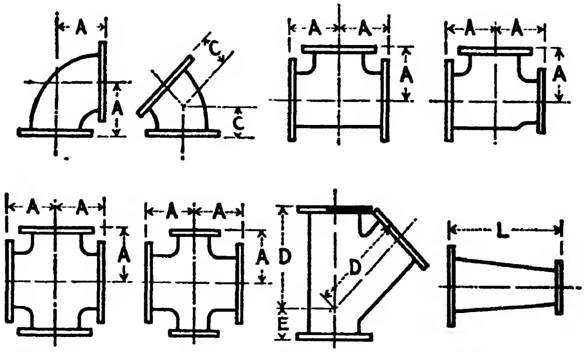


FIG. 184.—Flanged fittings for pipes (Table 85). See also Table 100, p. 382.

Table 85. Flanged Fittings for Pipes*

(Dimensions in inches)

Diam	Center to face A	Center to face C	Center to face D	Center to face E	Face to face L
①	②	③	④	⑤	⑥
3	$3\frac{5}{8}$	$2\frac{1}{2}$	9	$2\frac{1}{2}$	. . .
4	$4\frac{3}{8}$	$2\frac{15}{16}$	11	$2\frac{3}{4}$	12
5	$5\frac{1}{4}$	$3\frac{1}{4}$	12	3	12
6	$6\frac{1}{4}$	$3\frac{3}{4}$	$13\frac{1}{2}$	$3\frac{1}{4}$	12
7	$7\frac{1}{4}$	$4\frac{1}{8}$	15	$4\frac{1}{4}$	12
8	$8\frac{1}{4}$	$4\frac{1}{2}$	17	5	22
9	$9\frac{1}{4}$	$5\frac{1}{16}$	$18\frac{1}{2}$	$5\frac{1}{4}$	22
10	$10\frac{1}{4}$	$5\frac{7}{16}$	21	$5\frac{1}{2}$	22
11	11	$5\frac{3}{4}$	$22\frac{1}{2}$	$5\frac{3}{4}$	22
12	$12\frac{1}{4}$	$6\frac{5}{16}$	24	6	22
13	13	$5\frac{1}{2}$	26	$6\frac{1}{4}$	22
14	14	6	27	$6\frac{1}{2}$	22
15	15	$5\frac{1}{2}$	$29\frac{1}{2}$	$6\frac{3}{4}$	22
16	16	$6\frac{11}{16}$	$31\frac{1}{2}$	7	22
18	$16\frac{1}{2}$	$8\frac{1}{2}$	35	$7\frac{1}{2}$	32
20	18	$9\frac{1}{2}$	$38\frac{1}{2}$	8	32
22	20	10	41	9	32
24	22	11	44	10	32
26	23	13			42
28	24	14			42
30	25	15	..		42
32	26	16	..	.	42
34	27	17	.		42
36	28	18		. .	42
40	30	20	.		40

Center to face dimensions on other specials: On reducing outlets to tees, crosses and Y's—no change, on increasing outlets to tees—same as respective standards, on increasing outlets to crosses—determined by standard of largest opening

* Spiral Pipe Standard For 1914 standard of National Ass'n Master Steam & Hot Water Fitters, see pp. 382, 383

Trade names of fittings in Fig 184, reading in normal order, are ¼ bend (or elbow), ⅛ bend (or elbow), tee, reducing tee, cross, reducing cross, Y-branch, reducer (or increaser)

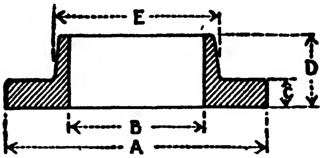


FIG. 185.—Shrink flange (Table 86).

Table 86. Standard (A. S. M. E.) Shrink Flanges, Forged and Rolled Steel

(Dimensions in inches)

Nominal diam	Outside diam A	Bore diam B	Thick-ness C	Depth of hub D	Diam. of hub E	No. of bolts	Size of bolts	Diam. bolt circle
1	2	3	4	5	6	7	8	9
4	9	4 3/8	15/16	2 3/16	5 3/4	8	5/8	7 1/2
4 1/2	9 1/4	4 7/8	15/16	2 1/4	6 1/8	8	3/4	7 3/4
5	10	5 7/16	15/16	2 5/16	6 7/8	8	3/4	8 1/2
6	11	6 1/2	1	2 7/16	7 7/8	8	3/4	9 1/2
7	12 1/2	7 1/2	1 1/16	2 1/2	9	8	3/4	10 3/4
8	13 1/2	8 1/2	1 1/8	2 5/8	10	8	3/4	11 3/4
9	15	9 1/2	1 1/8	2 3/4	11 1/8	12	3/4	13 1/4
10	16	10 5/8	1 3/16	3	12 1/4	12	7/8	14 1/4
12	19	12 5/8	1 1/4	3 3/8	14 1/2	12	7/8	17
14	21	13 7/8	1 3/8	3 3/8	15 7/8	12	1	18 3/4
15	22 1/4	14 7/8	1 3/8	3 1/2	16 7/8	16	1	20
16	23 1/2	15 7/8	1 7/16	3 5/8	18	16	1	21 1/4
18	25	17 7/8	1 9/16	3 7/8	20 1/8	16	1 1/8	22 3/4
20	27 1/2	19 7/8	1 11/16	4 1/8	22 1/4	20	1 1/8	25

Shrink flange with deep hub is made to comply with A. S. M. E standard for cast-iron flanges. They are made from best quality open-hearth steel, and may be shrunk and peened on heavy pipe without danger of cracking These flanges make reliable joints for medium- and high-pressure work. Flanges are furnished smooth forged to above dimensions except allowance on face for finishing. See more extensive table, p. 381 (American Spiral Pipe Works)

Table 86a.—Extra Heavy High Hub Flanges, Forged and Rolled Steel

(Dimensions in inches)

Nominal size	Outside diam. A	Bore, diam B	Thick-ness C	Depth of hub D	Diam of hub E	No. of bolts	Size of bolts	Diam bolt circle
4	10	4 3/8	1 1/4	3 1/4	5 3/4	8	3/4	7 7/8
4 1/2	10 1/2	4 7/8	1 1/4	3 1/4	6 1/4	8	3/4	8 1/2
5	11	5 7/16	1 1/4	3 1/4	7	8	3/4	9 1/4
6	12 1/2	6 1/2	1 1/4	3 1/4	7 13/16	12	3/4	10 5/8
7	14	7 1/2	1 5/16	3 3/8	9 1/8	12	7/8	11 7/8
8	15	8 1/2	1 3/8	3 1/2	10 5/16	12	7/8	13
9	16	9 1/2	1 7/16	3 5/8	11 3/8	12	7/8	14
10	17 1/2	10 5/8	1 1/2	3 3/4	12 5/8	16	7/8	15 1/4
11	18 3/4	11 5/8	1 9/16	3 7/8	13 5/8			
12	20	12 5/8	1 5/8	4	14 3/4	16	7/8	17 3/4
14	22 1/2	13 7/8	1 3/4	4 1/4	16 5/16	20	7/8	20
15	23 1/2	14 7/8	1 13/16	4 1/2	17 1/2	20	1	21
16	25	15 7/8	1 7/8	4 3/4	18 1/2	20	1	22 1/2
18	27	17 7/8	2	5	20 5/8	24	1	24 3/4
20	29 1/2	19 7/8	2 1/8	5 1/2	22 3/4	24	1 1/8	26 3/4

Flanges are furnished smooth forged to above dimensions, except allowance on face for finishing. Thickness may be increased as desired High hub flanges are not suited for threading, as bore is too large

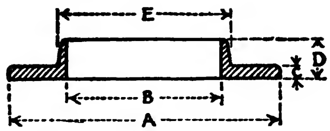


FIG. 186.—Flange for riveted steel pipe (Table 87).

Table 87. Flanges for Riveted Pipe

FORGED STEEL.—A. S. M. E. STANDARD DIAMETER AND DRILLING

(Dimensions in inches)

Nominal size, in.	Outside diam A	Actual bore B	Thickness C	Depth of hub D	Diam of hub E	No. of bolts	Size of bolts	Diam. bolt circle
①	②	③	④	⑤	⑥	⑦	⑧	⑨
4	9	$4\frac{3}{16}$	$\frac{9}{16}$	$1\frac{9}{16}$	$4\frac{13}{16}$	8	$\frac{5}{8}$	$7\frac{1}{2}$
5	10	$5\frac{3}{16}$	$\frac{9}{16}$	$1\frac{11}{16}$	$5\frac{7}{8}$	8	$\frac{3}{4}$	$8\frac{1}{2}$
6	11	$6\frac{3}{16}$	$\frac{9}{16}$	$1\frac{11}{16}$	$6\frac{7}{8}$	8	$\frac{3}{4}$	$9\frac{1}{2}$
7	$12\frac{1}{2}$	$7\frac{3}{16}$	$\frac{9}{16}$	$1\frac{13}{16}$	$7\frac{7}{8}$	8	$\frac{3}{4}$	$10\frac{3}{4}$
8	$13\frac{1}{2}$	$8\frac{3}{16}$	$\frac{5}{8}$	$1\frac{7}{8}$	$8\frac{7}{8}$	8	$\frac{3}{4}$	$11\frac{3}{4}$
9	15	$9\frac{1}{4}$	$\frac{5}{8}$	$1\frac{7}{8}$	$9\frac{15}{16}$	12	$\frac{3}{4}$	$13\frac{1}{4}$
10	16	$10\frac{1}{4}$	$\frac{11}{16}$	$1\frac{15}{16}$	$10\frac{15}{16}$	12	$\frac{7}{8}$	$14\frac{1}{4}$
12	19	$12\frac{1}{4}$	$\frac{3}{4}$	2	$12\frac{15}{16}$	12	$\frac{7}{8}$	17
14	21	$14\frac{1}{4}$	$\frac{3}{4}$	$2\frac{3}{4}$	$15\frac{1}{16}$	12	1	$18\frac{3}{4}$
16	$23\frac{1}{2}$	$16\frac{1}{4}$	$\frac{3}{4}$	$2\frac{3}{4}$	17	16	1	$21\frac{1}{4}$
18	25	$18\frac{5}{16}$	$\frac{3}{4}$	$2\frac{3}{4}$	$19\frac{1}{16}$	16	$1\frac{1}{8}$	$22\frac{3}{4}$
20	$27\frac{1}{2}$	$20\frac{5}{16}$	$\frac{7}{8}$	$3\frac{1}{8}$	$21\frac{1}{16}$	20	$1\frac{1}{8}$	25
22	$29\frac{1}{2}$	$22\frac{3}{8}$	$\frac{7}{8}$	$3\frac{1}{8}$	$23\frac{3}{16}$	20	$1\frac{1}{4}$	$27\frac{1}{4}$
24	32	$24\frac{3}{8}$	$\frac{7}{8}$	$3\frac{1}{8}$	$25\frac{3}{16}$	20	$1\frac{1}{4}$	$29\frac{1}{2}$

Larger sizes can be furnished on special order. When quotations are wanted on flanges with holes in hub for rivets state size and spacing. Flanges are smooth forged to above dimensions; shipped plain without holes unless ordered otherwise.

RIVETED PIPE MFRS' STANDARD

Nominal size, in.	Outside diam. A	Actual bore B	Thickness C	Depth of hub D	Diam of hub E	No of bolts	Size of bolts	Diam bolt circle
①	②	③	④	⑤	⑥	⑦	⑧	⑨
3	6	$3\frac{3}{16}$	$\frac{5}{16}$	$1\frac{1}{2}$	$3\frac{9}{16}$	4	$\frac{7}{16}$	$4\frac{3}{4}$
4	7	$4\frac{3}{16}$	$\frac{5}{16}$	$1\frac{1}{2}$	$4\frac{9}{16}$	8	$\frac{7}{16}$	$5\frac{15}{16}$
5	8	$5\frac{3}{16}$	$\frac{5}{16}$	$1\frac{1}{2}$	$5\frac{9}{16}$	8	$\frac{7}{16}$	$6\frac{15}{16}$
6	9	$6\frac{3}{16}$	$\frac{3}{8}$	$1\frac{1}{2}$	$6\frac{9}{16}$	8	$\frac{1}{2}$	$7\frac{7}{8}$
7	10	$7\frac{3}{16}$	$\frac{3}{8}$	$1\frac{5}{8}$	$7\frac{9}{16}$	8	$\frac{1}{2}$	9
8	11	$8\frac{3}{16}$	$\frac{3}{8}$	$1\frac{3}{4}$	$8\frac{5}{8}$	8	$\frac{1}{2}$	10
9	13	$9\frac{1}{4}$	$\frac{3}{8}$	$1\frac{3}{4}$	$9\frac{11}{16}$	8	$\frac{1}{2}$	$11\frac{1}{4}$
10	14	$10\frac{1}{4}$	$\frac{3}{8}$	$1\frac{7}{8}$	$10\frac{3}{4}$	8	$\frac{1}{2}$	$12\frac{1}{4}$
11	15	$11\frac{1}{4}$	$\frac{7}{16}$	$1\frac{7}{8}$	$11\frac{3}{4}$	12	$\frac{1}{2}$	$13\frac{3}{8}$
12	16	$12\frac{1}{4}$	$\frac{7}{16}$	$1\frac{7}{8}$	$12\frac{3}{4}$	12	$\frac{1}{2}$	$14\frac{1}{4}$
13	17	$13\frac{1}{4}$	$\frac{7}{16}$	2	$13\frac{3}{4}$	12	$\frac{1}{2}$	$15\frac{1}{4}$
14	18	$14\frac{1}{4}$	$\frac{7}{16}$	2	$14\frac{3}{4}$	12	$\frac{1}{2}$	$16\frac{1}{4}$
15	19	$15\frac{1}{4}$	$\frac{9}{16}$	2	$15\frac{3}{4}$	12	$\frac{1}{2}$	$17\frac{7}{16}$
16	$21\frac{1}{4}$	$16\frac{1}{4}$	$\frac{5}{8}$	$2\frac{1}{4}$	$16\frac{3}{4}$	12	$\frac{1}{2}$	$19\frac{1}{4}$
18	$23\frac{1}{4}$	$18\frac{5}{16}$	$\frac{5}{8}$	$2\frac{1}{4}$	$18\frac{7}{8}$	16	$\frac{5}{8}$	$21\frac{1}{4}$
20	$25\frac{1}{4}$	$20\frac{5}{16}$	$\frac{5}{8}$	$2\frac{1}{2}$	$20\frac{7}{8}$	16	$\frac{5}{8}$	$23\frac{1}{8}$
22	$28\frac{1}{4}$	$22\frac{3}{8}$	$\frac{11}{16}$	$2\frac{1}{2}$	23	16	$\frac{5}{8}$	26
24	30	$24\frac{3}{8}$	$\frac{11}{16}$	$2\frac{1}{2}$	25	16	$\frac{5}{8}$	$27\frac{3}{4}$

Flanges are smooth forged to above dimensions; carried in stock with and without bolt holes. All flanges shipped plain unless otherwise ordered Above standard adopted by leading manu-

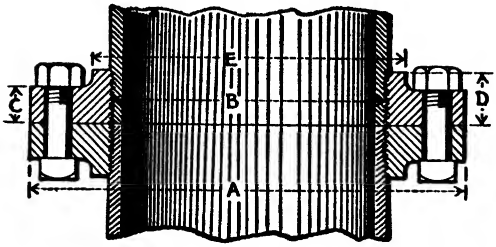


FIG. 187.—Standard companion flanges (Table 88).

Table 88. Standard Companion Flanges, Forged and Rolled Steel

(Dimensions in inches)

Nominal size, in	Outside diam A	Actual bore B	Thickness C	Depth of hub D	Diam. of hub E	No of bolts	Size of bolts	Diam. bolt circle
①	②	③	④	⑤	⑥	⑦	⑧	⑨
2	6	$2\frac{1}{8}$	$\frac{5}{8}$	1	$3\frac{1}{8}$	4	$\frac{5}{8}$	$4\frac{3}{4}$
$2\frac{1}{2}$	7	$2\frac{1}{2}$	$\frac{11}{16}$	$1\frac{1}{16}$	$3\frac{5}{8}$	4	$\frac{5}{8}$	$5\frac{1}{2}$
3	$7\frac{1}{2}$	$3\frac{1}{8}$	$\frac{3}{4}$	$1\frac{1}{8}$	$4\frac{5}{16}$	4	$\frac{5}{8}$	6
$3\frac{1}{2}$	$8\frac{1}{2}$	$3\frac{5}{8}$	$\frac{13}{16}$	$1\frac{3}{16}$	$4\frac{7}{8}$	4	$\frac{5}{8}$	7
4	9	$4\frac{1}{8}$	$\frac{15}{16}$	$1\frac{3}{16}$	$5\frac{3}{8}$	4	$\frac{3}{4}$	$7\frac{1}{2}$
$4\frac{1}{2}$	$9\frac{1}{4}$	$4\frac{5}{8}$	$\frac{15}{16}$	$1\frac{1}{4}$	$5\frac{13}{16}$	8	$\frac{3}{4}$	$7\frac{3}{4}$
5	10	$5\frac{1}{8}$	$\frac{15}{16}$	$1\frac{5}{16}$	$6\frac{7}{16}$	8	$\frac{3}{4}$	$8\frac{1}{2}$
6	11	$6\frac{3}{16}$	1	$1\frac{7}{16}$	$7\frac{9}{16}$	8	$\frac{3}{4}$	$9\frac{1}{2}$
7	$12\frac{1}{2}$	$7\frac{3}{16}$	$1\frac{1}{16}$	$1\frac{1}{2}$	$8\frac{5}{8}$	8	$\frac{3}{4}$	$10\frac{3}{4}$
8	$13\frac{1}{2}$	$8\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{5}{8}$	$9\frac{11}{16}$	8	$\frac{3}{4}$	$11\frac{3}{4}$
9	15	$9\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$10\frac{5}{8}$	12	$\frac{3}{4}$	$13\frac{1}{4}$
10	16	$10\frac{5}{16}$	$1\frac{3}{16}$	$1\frac{7}{8}$	$11\frac{15}{16}$	12	$\frac{7}{8}$	$14\frac{1}{4}$
12	19	$12\frac{5}{16}$	$1\frac{1}{4}$	$2\frac{1}{16}$	$14\frac{1}{8}$	12	$\frac{7}{8}$	17
14	21	$13\frac{1}{2}$	$1\frac{3}{8}$	$2\frac{3}{16}$	$15\frac{7}{16}$	12	1	$18\frac{3}{4}$

Flanges are furnished smooth forged to above dimensions, except allowance on face for finishing. Standard companion flanges, made to dimensions of A S M E standard cast-iron flanges, can be used with standard valves and fittings. They are sufficiently strong for high pressures and in many cases may be used in place of extra heavy standard The maker is prepared to furnish these flanges faced, drilled and threaded, every threaded flange being tested with Brigg's Standard gage to insure perfect fit of thread. (American Spiral Pipe Works.)

Galvanized Iron Pipe* shall be standard size, guaranteed wrought-iron pipe, galvanized, full weight, equivalent to pipe manufactured by A. M. Byers & Co., Pittsburgh. All pipe above $1\frac{1}{4}$ in. internal diam. shall be lap welded. All pipe less than and including $1\frac{1}{4}$ in. inside diam. may be butt welded. Weights shall not vary more than 5 per cent. from following:

Inside diam., in.	Weight per foot, lb.	Inside diam., in.	Weight per foot, lb.
$\frac{1}{2}$	0 84	$2\frac{1}{2}$	5 77
$\frac{3}{4}$	1 12	3	7 54
1	1 67	$3\frac{1}{2}$	9 05
2	3 66	4	10 72

Connections shall be made to main pipe by means of standard water-pipe clamp with threaded outlet. When possible connections shall be made to main line at a tapped plug. All threads of screw connections shall be unbroken and cut full depth, and before connections are made, well covered with steamfitter's cement. The pipe shall be laid with a cover of not less than 2 ft. and tested by hydrostatic pressure to 300 lb. per sq. in.

* Seattle, Wash., Standards.

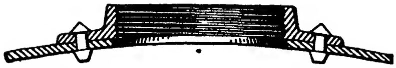


FIG. 188.—Tank flange (Table 89).

Table 89. Forged Steel Tank Flanges

Dimensions in inches

Standard tank flanges are made to meet a need for flanges lighter than the standard boiler flanges, and are especially suited to thin plate work, as they are readily drawn into place.

Nominal size, in	Outside diam.	Thickness	Depth of hub	Diam of hub
1	5	3/16	11/16	1 7/8
1 1/4	5 1/2	1/4	11/16	2 3/16
1 1/2	6	1/4	3/4	2 9/16
2	6 1/2	1/4	3/4	3 1/16
2 1/2	7 1/2	5/16	1	3 9/16
3	8	5/16	1 1/8	4 5/16
3 1/2	8 1/2	5/16	1 1/8	4 13/16
4	9 1/2	3/8	1 3/16	5 3/8
4 1/2	10	3/8	1 1/4	5 7/8
5	11	3/8	1 5/16	6 9/16
6	12	3/8	1 3/8	7 11/16

Threaded with standard taper thread. Following are smallest circles to which tank flanges are bent 1 in., 1 1/4-in., 1 1/2-in., 18-in circle. 2-in., 2 1/2-in., 3-in-, 30-in. circle. 3 1/2-in., 4-in., 4 1/2-in., 5-in. 6-in., 48-in. circle. Am. Spiral Pipe Works

Table 90. Steel or Wrought-iron Pipe, Standard Dimensions and Weights

See also page 346

Nominal inside diam., in	Actual inside diam., in	Actual* outside diam., in	Nominal thickness, in	Internal circumference, in.	External circumference, in.	Length of† pipe per sq. ft. of inside surface, ft.	Length† of pipe per sq. ft. of outside surface, ft
1/8	0 270	0 405	0 068	0 848	1.272	14 15	9 44
1/4	0 364	0 54	0 088	1 144	1 696	10.50	7 075
3/8	0 494	0 675	0 091	1 552	2 121	7.67	5 657
1/2	0 623	0.84	0 109	1 957	2 652	6 13	4 502
3/4	0 824	1.05	0 113	2 589	3 299	4.635	3 637
1	1 048	1 315	0 134	3 292	4 134	3.679	2.903
1 1/4	1 380	1 66	0 140	4 335	5 215	2.768	2.301
1 1/2	1.611	1 9	0 145	5 061	5 969	2.371	2.01
2	2.067	2 375	0 154	6 494	7 461	1.848	1 611
2 1/2	2.468	2 875	0 204	7 754	9 032	1 547	1.328
3	3.067	3 5	0 217	9 636	10 996	1 245	1 091
3 1/2	3.548	4 0	0 226	11 146	12 566	1.077	0 955
4	4.026	4 5	0 237	12 648	14 137	0.949	0 849
4 1/2	4.508	5 0	0 246	14.153	15 708	0 848	0 765
5	5.045	5 563	0 259	15 849	17 475	0.757	0.629
6	6.065	6.625	0 280	19 054	20 813	0 63	0 577
7	7.023	7.625	0 301	22 063	23 954	0 544	0 505
8	7 982	8 625	0 322	25 076	27.096	0 478	0 444
9	9 001	9 688	0 344	28 277	30 433	0 425	0 394
10	10 019	10 75	0 366	31 475	33 772	0.381	0 355
11	11 0	11 75	0 375	34 55	36 91	0 34	0 32
12	12 0	12 75	0.375	37 70	40 05	0 32	0.30

* Useful for figuring clearance

† Useful for figuring heat losses in power plants, heating surface of boiler tubes, etc

Table 90. Steel or Wrought-iron Pipe, Standard Dimensions and Weights.—*Continued*

See also page 345

Nominal inside diam., in	Internal area, sq. in.	External area, sq. in.	Length of pipe containing 1 cu. ft., feet	Nominal* weight per ft., lbs.	No. of threads per in. of screw	Contents in gal. per ft.	Size drills, in., to be used for tapping
1/8	0.0572	0.129	2500.0	0.243	27	0.0006	21/64
1/4	0.1041	0.229	1385.0	0.422	18	0.0026	29/64
3/8	0.1916	0.358	751.5	0.561	18	0.0057	19/32
1/2	0 3048	0.554	472.4	0.845	14	0.0102	23/32
3/4	0.5333	0.866	270.0	1.126	14	0.0230	15/16
1	0.8627	1.357	166.9	1.67	11½	0.0408	1 3/16
1¼	1.496	2.164	96 25	2.258	11½	0.0638	1 15/32
1½	2.038	2.835	70 65	2.694	11½	0.0918	1 23/32
2	3 355	4.430	42 36	3.667	11½	0 163	2 3/16
2½	4 783	6.491	30 11	5.773	8	0.255	2 11/16
3	7.388	9 621	19.49	7 547	8	0.367	3 5/16
3½	9 887	12.566	14 56	9.055	8	0.500	3 13/16
4	12 730	15.904	11.31	10.728	8	0.653	
4½	15 939	19 635	9.03	12 492	8	0 826	
5	19 990	24 299	7.20	14.564	8	1.020	
6	28 889	34 471	4.98	18.767	8	1.469	
7	38 737	45 663	3.72	23.410	8	2.00	
8	50.039	58 426	2.88	28 348	8	2.61	
9	63 633	73 715	2.26	34.077	8	3.30	
10	78 838	90 762	1.80	40 641	8	4.08	
11	95 03	108 43	1.50	45.0	8	4.93	
12	113 09	127 67	1 27	48 98	8	5 87	

Table 91. Dimensions and Weights of Large Outside Diameter Wrought-iron Pipe.†

Thickness, in.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
Outside diam. of pipe, in	Weight per ft, lbs	Weight per ft., lbs.	Weight per ft, lbs	Weight per ft, lbs.	Weight per ft, lbs.	Weight per ft, lbs	Weight per ft, lbs.	Weight per ft, lbs	Weight per ft, lbs.
14	36 75	45.72	54 61	63 42	72 16	80 80	89 36	97.84	106.2
15	39 42	49.06	58 62	68 10	77 50	86 81	96.03	105.2	114.2
16	42 09	52 40	62 63	72 78	82.85	92.83	102.7	112.5	122.2
18	47 44	59 08	70 65	82 14	93.54	104.8	116 1	127.2	138.3
20	52.78	65.76	78 67	91 49	104.2	116.9	129.4	141.9	154.3
22	. .	72.44	86 68	100 8	114.9	128.9	142.8	156.6	170 3
24		79.13	94 70	110 2	125.6	140.9	156.2	171.3	186 3
26		. .	102 7	119 5	136.3	152.9	169.5	186.0	202.4
28	.		110.7	128 9	147.0	165.0	182.9	200.7	218.4
30	. . . .		. .	138 2	157.7	177.0	196.3	215.4	234.4

* Add 2 per cent. for weight of steel pipes.
† American Spiral Pipe Works.

Table 92. Lap-welded Artesian-well Casing

Nominal inside diam., in.	Actual outside diam., in., not including couplings	Nominal weight per ft., lbs.	No. of threads per in. of screws
2	2¼	2 22	14
2¼	2½	2 82	14
2½	2¾	3 13	14
2¾	3	3 45	14
3	3¼	4 10	14
3¼	3½	4 45	14
3½	3¾	4 78	14
3¾	4	5 56	14
4	4¼	6 00	14
4¼	4½	6 36	14
4¼	4½	9 38	14
4½	4¾	6 73	14
4½	4¾	9 39	14
4¾	5	7 80	14
5	5¼	8 20	14
5	5¼	9 86	14
5	5¼	12 80	11½
5	5¼	15.88	11½
5 3/16	5½	8 62	14
5 3/16	5½	12 49	11½
5⅝	6	10 46	14
5⅝	6	12 04	11½
5⅝	6	14 20	11½
5⅝	6	16 70	11½
6¼	6⅝	11 58	14
6¼	6⅝	13 32	14 & 11½
6¼	6⅝	17 02	11½
6⅝	7	12 34	14
6⅝	7	17 51	11½ & 10
7¼	7⅝	13 55	14
7⅝	8	15 41	11½
7⅝	8	20 17	11½
8¼	8⅝	16 07	11½
8¼	8⅝	20 10	11½
8¼	8⅝	24 38	11½ & 8
8⅝	9	17 60	11½
9⅝	10	21 90	11½
10⅝	11	26 72	11½
11⅝	12	30 35	11½
12½	13	33 78	11½

WELDED STEEL PLATE PIPES

Methods and Sizes. Continental Iron Works, Brooklyn, process consists in lapping the edges of the plates by a proper amount, heating to welding temperature with a specially designed gas furnace, and rolling between two rolls, one of which is shoved against the other by a hydraulic ram exerting 25 tons pressure. Roller is preferred to hammer because quicker and effective over a larger area; 50 lin. ft. of ½-in. plate welded in 10 hrs. Welding up to 1-in. plate is feasible. No flux is used. Company claims that joint thus made is as strong as the rest of the plate; it has no record of a break at a weld; an 18-in. steam pipe burst under 1750 lbs. test pressure, split 4 in. from the seam. This company fabricated two steel pipe-lines: 1100 ft. of 57-in., Guanajuata

Power & Electric Co., Zamora, Mex., thickness, $\frac{3}{8}$ to $\frac{7}{16}$ in.; 4790 ft. of 43-in. pipe, $\frac{11}{16}$ in. thick, Yukon Consolidated Gold Field Co., 500 lbs. working pressures. On the Yukon, joints were bell-and-spigot, tapered, and drilled for rivets after being put together. Bends, up to 4°, were made by beveling plates of adjacent sections, and welding. Two makes,* one American and one German, were used; latter was not entirely satisfactory. One section cracked along a weld for 10 ft. on being dropped into trench; this was patched and reinforced, and gave no more trouble. Another section burst under 835 ft. test head, due to flaw in metal.

Forge-welded steel plate pipes, in lengths up to 50 or 60 ft., have been made in Germany for a number of years and used throughout Europe and in more distant parts of world, for water supply and power. A similar process was recently installed by M. W. Kellogg Co., N. Y. City, for sizes up to 72 in., thickness up to $1\frac{1}{2}$ in.

Failures of Large Sizes. The claim that lap-welded pipe is stronger and more reliable than riveted pipe is disputed. Of an installation of four welded pipes two failed at the weld, one after being in service 7 yrs. Rupture showed the weld blistered nearly the entire length, and for 9 in. a flaw $\frac{2}{3}$ through the metal. At the time of break, very cold water was being drawn; it was supposed that contraction caused extra stress at the weld (pipe was flattened at joint). Numerous lengths of tested pipe have had to be discarded; a test of a few minutes is not sufficient. It is advisable to have as few cross connections as possible, and these, where feasible, normally closed. At another plant, of two lap-welded pipes, one opened at a weld shortly after installation; after 7 yrs. another section opened at the weld; inspection showed an imperfect weld and shearing of the outer portion of the steel from the inner along the neutral axis. At a third plant of two lines, partly 30 in., five sections were rejected in the field for defective welds; examination by drilling showed in cases less than half of the metal in contact. In another case, 24-in. pipe for a maximum head of 1068 ft. tested at shop, one section was found to have opened 5 ft. along the weld; after being in service 2 weeks another section opened at the weld. (Trans. A. S. C. E, Vol. 58, 1907, pp. 57–59.)

SPECIFICATION FOR STANDARD WELDED PIPE, NATIONAL TUBE CO.

Surface Inspection. Pipe shall be reasonably straight and free from blisters, cracks or other injurious defects. Liquor marks incidental to manufacture of lap-welded pipe will not be considered surface defects. Pipe shall not vary more than 1 per cent. either way from perfectly round or true to standard outside diam. except small sizes, where variation of $\frac{1}{64}$ in. will be accepted. Pipe shall not vary more than 5 per cent. either way from standard weight.

Threading and Reaming. Where required, pipe shall have good Briggs standard threads, which will make tight joints when tested by hydrostatic pressure at the mill. Thread shall not vary more than $1\frac{1}{2}$ turns either way when tested with Pratt & Whitney Briggs standard gage. All burrs at end shall be removed.

Internal Pressure Test. Following test pressures will be applied to respective sizes of standard butt- and lap-weld pipe:

* Trans A. S. C. E., Vol 78, 1915, W W Edwards.

Nominal size, inches	Weld	Test pressure, pounds
$\frac{1}{8}$ to 2	Butt	700
$2\frac{1}{2}$ and 3	Butt	800
Up to 8	Lap	1000
9 and 10	Lap	900
11 and 12	Lap	800
13 and 14	Lap	700
15	Lap	600

NOTE For 8, 10 and 12 in., which have more than one weight as standard, test pressure for heaviest is given.

Mannesmann Seamless Steel Tubes, made at Swansea, England, under German patents, have been used for 2 yrs. for distributing mains at Victoria,

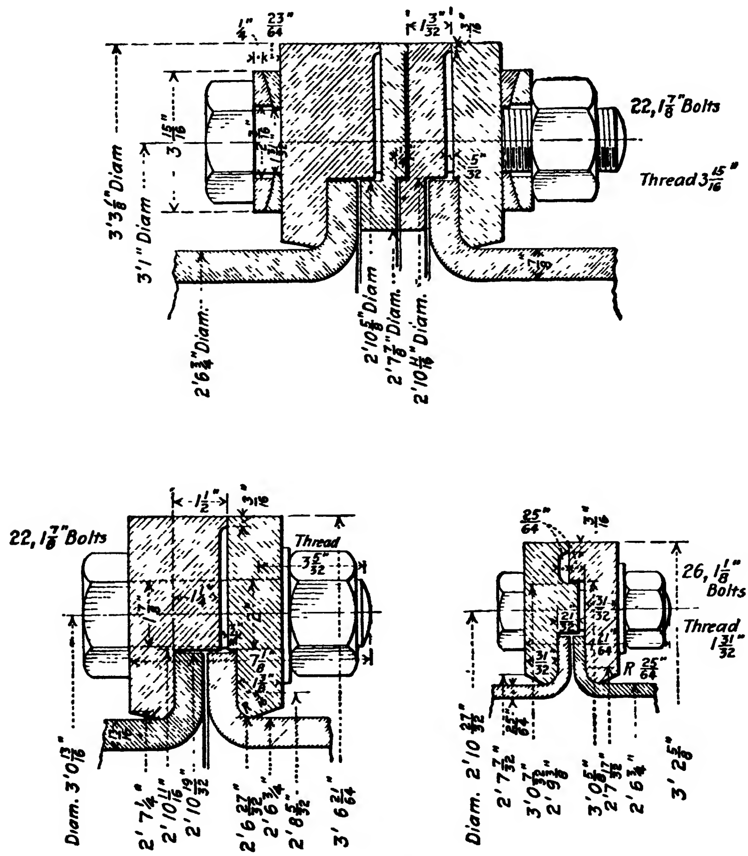


FIG. 189.
(T. A. S. C. E., Vol. 58, 1907, p. 42.)

B. C. Tubes came in 28-ft. lengths; coated with asphalt, wrapped with jute, and again coated with asphalt. Prices per ft. paid by city for pipe delivered along trench: 12-in., $1.59; 8-in., $0.72; 6-in., $0.52; 4-in., $0.295. Specials, $0.21 per lb. Pressure not given. (E. N., July 18, 1912, p. 119.)

Joints for High-pressure Steel Pipes. For Necaxa, Mexico, power plant, 30-in. pipes to carry water under 1450-ft. head were forged complete with flanges from one sheet of steel for each 30-ft. section. Longitudinal seams were lap-welded. Pipes were tested in shop to 1¾ times maximum working head. Maximum velocity of water was 18 ft. Special joints used are shown in Fig. 189. Pipes were made by Actien Gesellschaft Ferrum, Kottowitz, Germany. Rubber gaskets were used.—F. S. Pearson and F. O. Blackwell, Engineers. (T. A. S. C. E., Vol. 58, 1907, p. 42.)

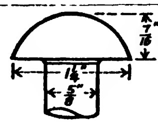
WROUGHT-IRON RIVETED PIPE* SPECIFICATIONS†

These specifications are given at length because of the care with which they were prepared by the company's former chief engineer, Herrmann Schussler, and the successful results achieved. Furthermore, there appears to be an increasing conviction that genuine wrought iron is, at least under some conditions, much more resistant to corrosion than steel.

Manufacture of Flanged Boiler Iron. Take charcoal iron blooms rolled into bars about 14 in. wide and 3 to 4 ft. long, about 1 in. thick, for bottom of pile. This bloom to be made of iron only, melted in charcoal fire and hammered into a bloom, then rolled into above wide flat bar. On this slab bars and best scrap, both of charcoal iron, are closely and evenly packed crosswise and with flush or even top. On top of this pile, another 3 to 4-ft. × 14-in. × 1-in. slab, as above, is placed, making a pile 8 to 10 in. thick, or high. Thereupon this pile is heated to welding heat, put through rolls, and welded down to 4½-in. thickness, more or less, *across* grain of cover. Then pile is reheated and rolled out to width of finished plate *with* grain of cover. Then slab is quickly turned at right angles and rolled against grain of cover to full length of finished plate. Scrap from shearing plates goes to make up next piles. Tensile strength shall be between 48,000 and 50,000 lbs. per sq. in. lengthwise with plate, 2 per cent. variation allowed below lower and above upper limit. Tensile strength crosswise with plate shall not be less than 40,-000 lbs. Elastic limit about 32,000 and not less than 30,000 lbs. per sq. in. Elongation 15 to 20 per cent. in 8-in. section. Reduction of area not less than 25 per cent. Thickness shall not vary more than 3 per cent. above or below given standard. Each plate shall be sheared true to dimensions, with sides and ends straight and rectangular to each other. Shipping weight of plates shall not be more than 8 per cent. above nor 3 per cent. below required weight of plate computed at 40 lbs. per sq. ft. 1 in. thick, and thickness of plate at sheared edges shall not be less than 97 per cent. of thickness specified. Each plate shall be flat, smooth and even, of even thickness throughout, and free from warping, buckles, cracks, splits, flaws, rust and all other defects. The iron shall be of American manufacture, close-grained, tough and thoroughly pliable while cold, also allowing cold scarfing to a fine edge at laps, without splitting or cracking, and shall not crack between rivet holes, between holes and edge of plate while being rolled, or while rivets are being driven. All plates showing flaws of any kind or exhibiting a hard and brittle character, and that do not in every way meet above requirements, will be rejected. Plates may vary *in width* $\frac{5}{16}$ in., but shall be not less than $\frac{1}{16}$ in. less than 60 in., nor more than 60¼ in. *In length*, they shall be not less than ⅛ in. below nor more than ¼ in. above lengths specified. Contractor shall be responsible for storing plates under a tight roof in mill premises and in tight box cars, without exposure to moisture of any kind.

* See also page 300. † Spring Valley Water Co., San Francisco

Making 54-in. Pipe. Plates and rivets of which pipe is to be made will be delivered at place of manufacture in San Francisco. The following table shows dimensions of plates and rivets.

Plates	Thickness, in.	Weights, lbs. per sq. ft.		Size of plate, in.
	$\frac{9}{32}$	11		58×175
	$\frac{9}{32}$	11		58×176½
		Size of body, in.		Rivet head
		Diam.	Length	
Rivets	Seam	$\frac{5}{8}$	$1\frac{5}{8}$	
	Lap	$\frac{5}{8}$	$1\frac{1}{4}$	

During manufacture of pipes, all plates and rivets shall be kept under roof and cover, and in no way exposed to rain or fogs. Pipe is to consist of large and small straight courses. Plates for large courses, 58 in. wide by 176½ in., are to be trimmed to exact size, with opposite sides parallel to each other and corners rectangular. Plates for small courses, 58 in. wide by 175 in., are to be trimmed so that when punched and riveted into courses they fit tightly in large courses. Rivet holes are to be punched as follows: Center to center in each row of straight seams, 2¼ in.; center to center between two rows of straight seams, 1¼ in.; center to center, round seam, 1⅞ in.; center of seam to edge of sheet, 1¼ in.; diam. of rivet holes, $\frac{11}{16}$ in. Where, at end of each course, lap falls between two thicknesses of iron, plates shall be drawn to a fine edge, through which edge, upon riveting courses together, one rivet of round seam shall be driven to insure absolute tightness. Each plate shall be rolled to a perfect cylinder of required diam. All punching of plates shall be done by automatic and accurate multiple punching machine and not through a frame or templet—opposite lines of rivet holes to be absolutely straight and parallel to each other and spaces between respective holes evenly divided.

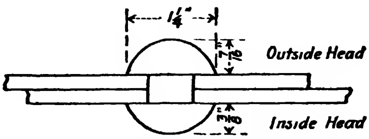


Fig. 190.

All riveting in shop shall be done with hot rivets and by hydraulic machinery, exerting a slow pressure of not less than 20,000 lbs. on each rivet head. Prior to driving rivet, plates shall be pressed together by a slow squeeze from same machine. Head of rivet on inside and outside of pipe is to be formed by a cup in the die, and inside rivet head not to be higher than ⅜ in. and to receive, as nearly as possible shape shown in Fig. 190, while outside head may be $\frac{7}{16}$ in. high in center.

At each junction of straight and round seam where three thicknesses of iron come together, lap rivets are to be used. Straight seams are to be all on one side of each length of pipe, alternating to right and left not more than 1 ft. Each pipe length shall consist of six large and small straight courses, with a large course at one end and a small course at other. All round and straight seams and laps are to be thoroughly split and calked in first-class boiler-works fashion, while for 4 in. from all laps, all seams are to be chipped and calked. S. V. W. W. may test any length with water pressure up to 200 lbs. at expense of contractor. Pipe must be absolutely tight under pressure of 200 lbs. per sq. in. Small course shall be either

chipped or ground with emery wheel to bevel of 45°, so as to decrease friction of water in pipes. All punching, riveting, calking, etc., shall be done with best workmanship, so as to insure strength and absolute tightness. S. V. W. W. is to dip the pipe in asphaltum coating and to transport to and distribute along ditch. Space for asphaltum kettles and tanks, and rig and power for handling pipe while being dipped, and ample room for dry storage of pipe before dipping and for open air storage after dipping, shall be furnished by contractor.

Laying Pipe. Pipe ditch and necessary joint holes will be dug by S. V. W. W. Pipes are to be connected in ditch by inserting small course of one pipe into large course of other. Where pipe curves, strap joints are to be used, iron and rivets necessary for such joints to be furnished by S. V. W. W., strap iron being 8 in. in width and to be put on outside of pipe in two sections, with straight seams on sides, so as to form joints as perfectly as those required to be made in shop. Contractor is to roll, shear, punch and fit bands, and employ best of workmanship in this work and in scarfing, riveting, splitting, chipping and calking and other necessary work, as is required for pipe. The two straight seams of strap shall be chipped and calked, as well as round seam, for 4 in. on each side of laps. Where straps are used, lengths of pipe are to be so placed in ditch that ends of pipe butt together, or nearly so. Distance between ends of pipes at such bends, in widest part, is not to be

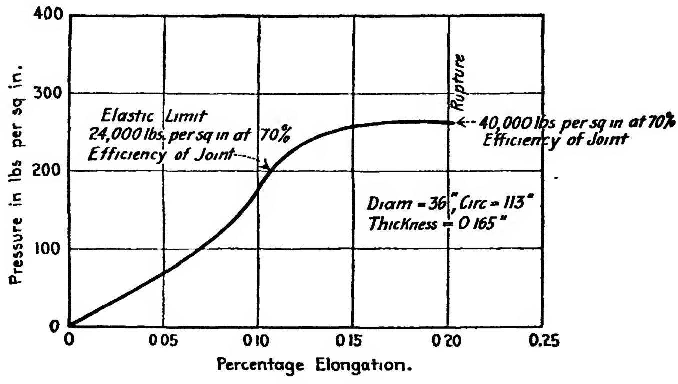


FIG. 191.

more than 3 in. Before straps are fitted and riveted to ends of pipe, these ends are to be carefully scraped and entirely cleaned of coating for 3 in. from each end, both in and outside of pipe, same as all other joints made in ditch, so that strap joints as well as other joints make a perfect union of iron to iron. Round seams of pipe, as well as seams of strap joints, shall be riveted with hot rivets, forming good, substantial heads, both in and outside of pipe, of shape and proportions shown on p. 351, care being taken that a rivet is placed through scarfed edges of plate at all laps, for which laps lap rivets are to be used as specified for pipe. Where curvature of pipes is so great that above strap joints are insufficient to make pipe follow such curves, same is to be accomplished by inserting one or more single courses, or such single courses intermingled with lengths of pipe. Contractor is to furnish manholes, blow-outs and air valves, and connect them with pipe in such places as chief engineer directs, each fitting being provided with a wrought iron ring $\frac{5}{16}$ in. by 3 in. on inside, hot rivets passing through it and iron of pipe and flanges of fitting—only hot rivets being used; edge of plate shall be chipped and calked against inside face of fitting; rivet-joints, as well as apparatus so attached, shall be perfectly

water-tight. Any plates or rivets injured or allowed to rust, or otherwise damaged by contractor while in his keeping, and before acceptance and dipping by S. V. W. W. shall be replaced at his expense.

Contractor shall provide dry storage for iron and rivets, so as to keep moisture, rain, fog, etc., entirely away from them. Contractor is required to manufacture, ready for dipping and transportation, for every working day after arrival of iron, not less than ——— feet of pipe, and is also required to lay and connect the same, after it has been dipped and transported by the S. V. W. W., in manner described and specified, at same rate per day as it was manufactured.

Tests of Strength and Stretch. Engineers of Spring Valley W. W., San Francisco, tested (June, 1911) 36-in. riveted wrought-iron pipe made under specifications like above, with results shown in Fig. 191. Elongation of the iron or circumferential stretch of the pipe, was measured by a tape wrapped around the pipe and held by special apparatus.

INGOT IRON PIPES

In Uncompahgre Valley, Colorado, U. S. Reclamation Service, in 1911, built a 26-in. pipe of "Ingot" iron sheets, since the alkali soils along the line have a strong corrosive action on ordinary iron and steel. This is believed to be the first extensive use for pipe lines. Specifications called for following properties: Ultimate tensile strength, not less than 48,000 nor more than 52,000 lbs. per sq. in.; elastic limit, not less than 35,000; carbon, not more than 0.01 per

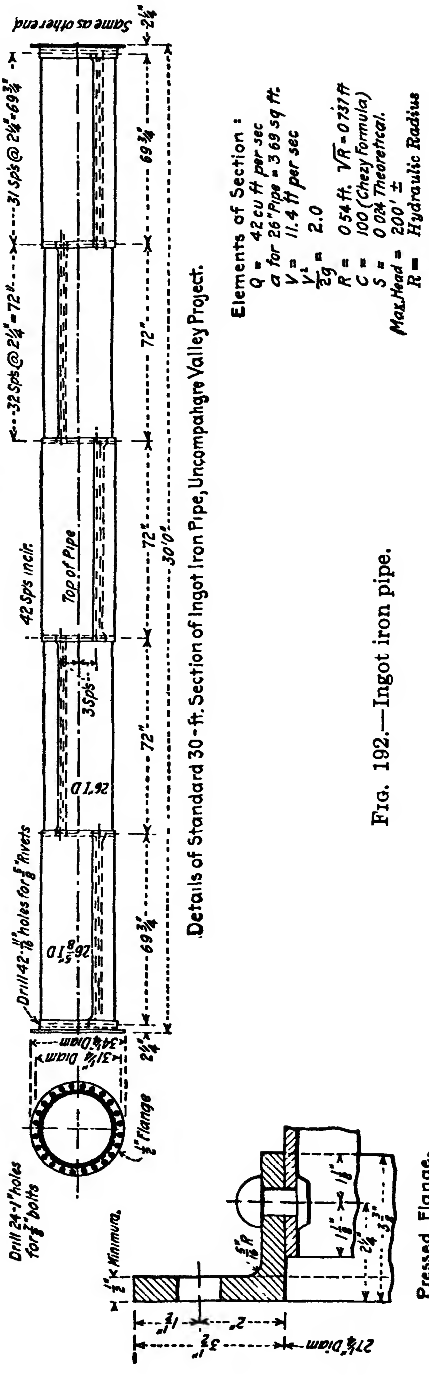


Fig. 192.—Ingot iron pipe.

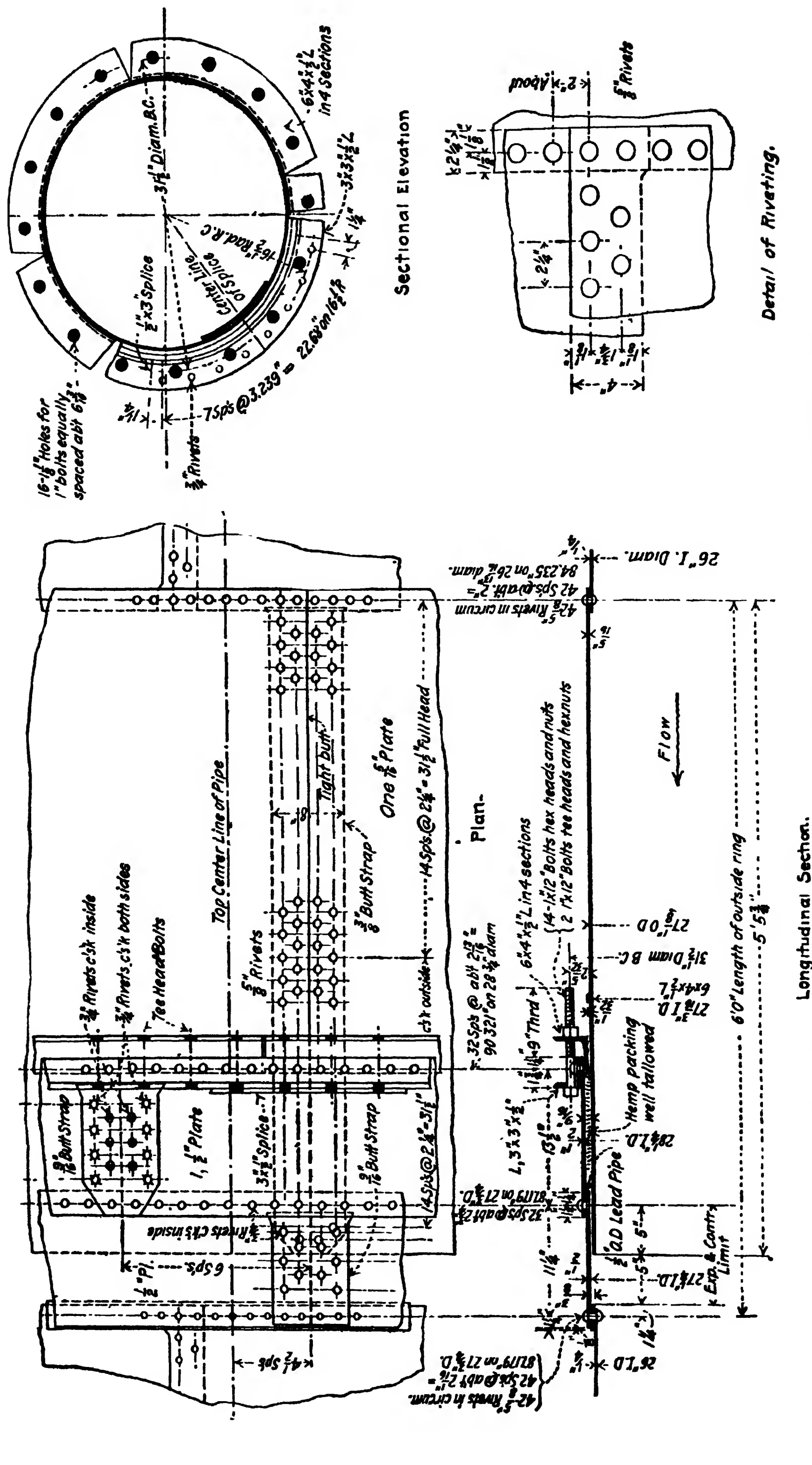


FIG. 193.—Ingot iron pipe, Uncompahgre Valley project, expansion joint.

cent.; manganese, 0.02; phosphorus, 0.005; sulphur, 0.02; oxygen, 0.03 per cent.; and trace of silicon. (E. R., July 8, 1911.)

Table 93. Results of 28 Tests of "Ingot" Iron Plates

	Max.	Min.	Ave.
Elastic limit, lbs. per sq. in.	46,750	22,800	34,840
Ultimate strength, lbs. per sq. in.	59,000	40,580	49,230
Elongation in 8 in.	37%	10%	26%
Reduction of area	88%	40%	68%
Carbon, per cent	0.02	0.01	0.012
Manganese, per cent	0.02	0.01	0.013
Phosphorus, per cent	0 011	0.001	0.0053
Sulphur, per cent	0.017	0.022	0.02
Oxygen, per cent	0.037	0.003	0.022

RIVETED PIPE LINES: HYDRAULICS AND MISCELLANY

Capacities of Riveted Steel Pipes* are less than those of cast-iron pipes of same diameters, other things being equal. One rule is to make a steel pipe, for equal capacity, of a diam. greater by twice the hight of the rivet heads, but some engineers require a greater difference. In 1910, for Portland, Me., bids were asked on 42-in. cast-iron and 48-in. steel as alternatives for the same line. For lock-bar pipe, capacity 10 to 12 per cent. greater than ordinary riveted pipe is claimed. Longitudinal joints have little effect on friction losses.

Table 94. Gagings of Northern Section of New Conduit, Rochester, N. Y. Waterworks, by Same Parties, at Different Times between Oct., 1895, and Nov., 1899

Date	Duration of experiment, hrs.	Observed hydraulic grade in 36-in. cast-iron pipe	Observed hydraulic grade in 38-in. riveted-steel pipe.	Observed mean velocity in 36-in. cast-iron pipe, ft. per sec.	Observed mean velocity in 38-in. riveted-steel pipe, ft. per sec.	Deduced coefficient C in $V = C \times \sqrt{Rs}$ for 36-in. cast-iron pipe	Deduced coefficient C in $V = C \times \sqrt{Rs}$ for 38-in. steel pipe	Mean age of conduit in service, yrs.
1	2	3	4	5	6	7	8	9
1895								
Oct. 17	5.35	0.0013876	0.0015866	4.204	3.876	129.45	109.35	1.2
Oct. 26	5.22	0.0015066	0 0016137	4.239	3.908	125.25	109 34	1 3
Nov. 7	6.38	0.0015034	0.0016180	4.234	3.904	125.25	109.07	1.3
1897								
Jul. 28	10.77	0.0022774	0.0016250	4.128	3.806	99.22	106.11	3.0
Nov. 11	7.92	0.0048187	0.0015315	4.045	3.730	66.84	107.11	3 3
Nov. 19	7.08	0.0048535	0.0015334	4.023	3.709	66.24	106.46	3.3
1898								
June 4	8.95	0.0043408	0 0015655	4.034	3.719	70.23	105.65	3.9
Dec. 21	8.00	0.0037591	0 0015531	4.026	3.712	75.32	105.86	4.4
1899								
Jul. 25.	8.00	0.0034455	0 0015349	4 084	3.765	79 79	108.00	5 0
Nov. 10	8 00	0 0032585	0.0015424	4 077	3 759	81.93	107.58	5.3

Corrosion. Uncoated steel plates corrode with varying degrees of rapidity, while coated ones do not corrode until the coating is penetrated. Electrolytic theory of corrosion is most satisfactory.† Electrolysis results from difference

* See also Fig. 319, page 570.

† Electrolytic theory: Two unlike metals in electrical contact and both in contact with a liquid which is able to attack the iron and cause it to go into solution as ferrous hydroxide which, in the presence of oxygen, is rapidly oxidised to ferric hydroxide; this precipitates as rust. A. S. Cushman, Bull. 30, Office of Public Roads.

of potential due to (1) segregation in steel, (2) enclosed pieces of slag, (3) slightly different qualities of metal in adjacent sheets, (4) mill scale on metal, (5) coating and (6) electrolyte in the water of the conduit, or in stagnant

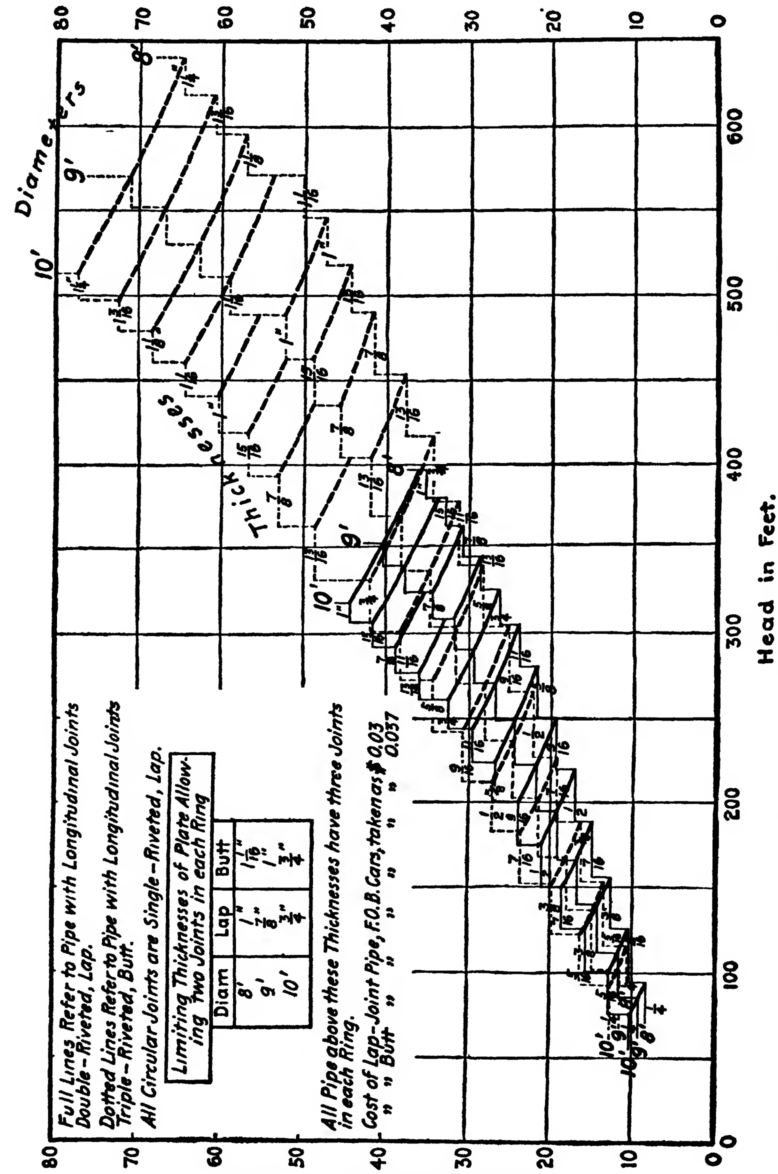


FIG. 194.—Cost-head diagram for large lap and butt-joint steel pipes. (New York Board of Water Supply Studies.)

water adjacent to the outside of plates. Electrolytic theory, insofar as it explains corrosion, also indicates a remedy. Metal surrounded by alkali cannot rust since acid hydrogen ions are prevented from coming into contact

with it. Mortar and concrete are decidedly alkaline; this explains their preservative effect on both iron and steel.

Life. Steel pipe siphons in 7.5 ft. diam., on the Weston aqueduct, laid in

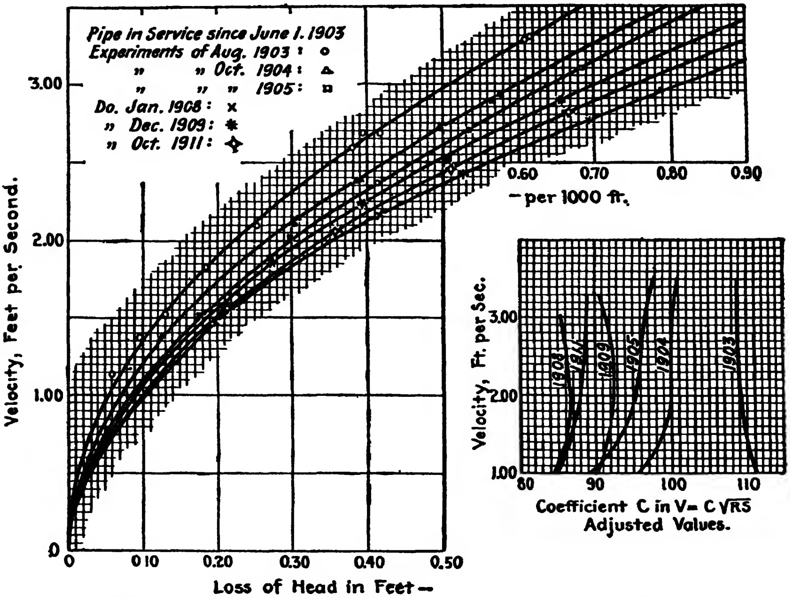


FIG. 195.—Jersey City Water Supply Co. 72-in. riveted steel pipe. Coefficients, velocity and loss of head.

About September, 1908, a plant for treating this water with chlorinated lime was installed at Boonton, N. J., and since that date water has been continuously so treated. Immediately following introduction of treatment, large masses of vegetable and other matters were discharged from pipe, and improvement in carrying capacity as indicated by experiments of January, 1908, and December, 1909, is undoubtedly due to this cause.

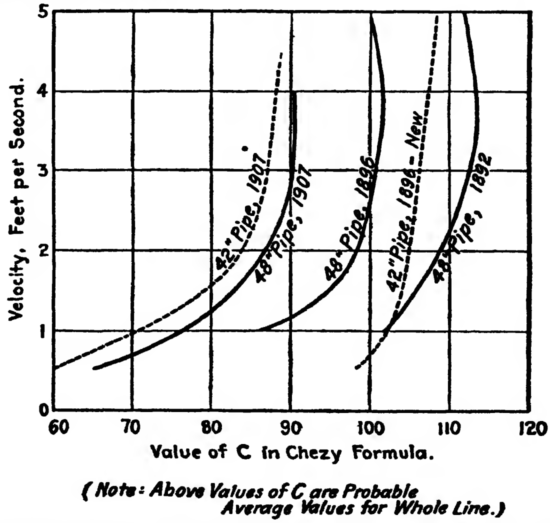


FIG. 196.—Riveted steel conduits, Newark, N. J. Comparative values of C in 1892, 1896, 1907.

1898, have given no trouble. Pipe arch bridge over Sudbury river (90-ft. span) has not leaked nor shown signs of distress. For Catskill aqueduct studies 35 yrs. were assumed as the renewal interval for steel pipe with the best dip coatings; so far as known, no steel main has been in use for that period. Steel pipes at Newark, N. J., about the oldest in eastern U. S., were laid in 1890–91 and in service in 1915. Electrolytic action may destroy steel pipe in much less than 35 yrs.

Growths. Organisms which lodge and develop in pipe may be animal or vegetable; they generally grow on the sides and the top, rarely on the bottom, as they appear to dislike the subsidence of fine silt apt to occur when the velocity is greatly reduced. Polyzoa, spongilla, algæ and fungi are found in dark chambers of reservoir gate-houses and in conduits and distributing pipes, but in the latter are usually restricted to the vicinity of the source or reservoir. In some cases these growths are found in pipes several miles from the head.

Leakage. General experience with steel mains is that they tend to grow tighter with age, unless perforations occur. A 48-in. steel main in Philadelphia, after being tested and recalked, had leakage of 7000 to 10,000 gals. per mile per 24 hrs. under pressure of 160 lbs. per sq. in. A 72-in. steel main 10.8 miles long, tested in sections, averaged 0.059 cu. ft. per sec. per mile leakage, pressure 0 to 59 lbs. = 38,000 gals. per mile per 24 hrs. Tests on some Brooklyn conduits resulted as follows: (1) 66-in. pipe, 3.2 mi. long, 33 per cent. riveted steel, remainder of lock-bar type; field joints every 30 ft.; leakage under 65 to 95 lbs. test pressure was 12,300 gals. per mile per day (3.91 gals. per lin. ft. of field joint per 24 hrs.). (2) 66-in. pipe, 30 per cent. riveted steel, remainder of lock-bar type; field joints every 30 ft. Leakage under pressure of 48 to 117 lbs. per sq. in. was 9800 gals. per day per mile (3.10 gals. per lin. ft. of field joint per 24 hrs.). See also p. 414.

Steel Pipe Deformation. Kuichling (New England Waterworks Ass'n, 1910) has seen earth fill 6 to 8 ft. deep reduce 10 per cent. the vertical diam. of 36 in. to 72 in. steel pipes, $\frac{1}{4}$ to $\frac{1}{2}$ in. thick. He concludes that ordinarily 5 or 6 ft. backfill will produce stresses near elastic limit. Stiffening rings of steel or concrete should be used on deep pipes.

A 42-in. steel pipe, Portland, Oregon, was flattened 4 in. by careless backfilling, causing uneven distribution of load. Tests showed no leakage under distortion of $8\frac{3}{4}$ in., although shortening of only $1\frac{5}{8}$ in. caused permanent set of $\frac{1}{2}$ in.—(E. R., Vol. 38, 1898, p. 554.)

Temperature Effect. On a 72-in. riveted steel main laid in Brooklyn, 1909, the lead was pulled 2 in. out of joints 9 in. deep at a valve, at night, by temperature effect on 3500 ft. of exposed line on either side, and forced back partly by day. Temperature ranged from about 38 to 60° F. On tangents usually the only force to be resisted by the circular seams is that due to temperature. Assuming a temperature variation of 45°, modulus of elasticity for steel, 30,000,000; tensile strength, 55,000 lbs. per sq. in.; elongation per degree, 1 ÷ 148,000, and factor of safety of 3, necessitates a joint having $(30{,}000{,}000 \times 45) \div (148{,}000 \times 55{,}000 \div 3) = 50$ per cent. efficiency. These temperature stresses cause shear in the rivets of field seams which generally determines the spacing of the rivets on single-riveted circular seams.

CHAPTER XVI

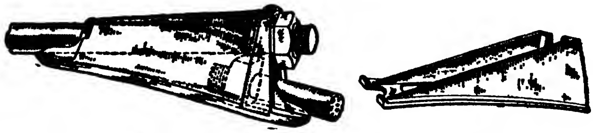
WOODEN PIPE

CONTINUOUS STAVE PIPE

Construction. Continuous stave wooden pipe is constructed in diam. 18 in. to 10 ft. and even larger, of selected lumber so milled that when assembled the staves form pipe of size desired. Staves of various lengths are laid side by side so that all joints are staggered. Each stave as laid is butted against one immediately preceding; saw kerfs, or slits, in ends receive thin metal tongue, thus making staves continuous. Staves are cinched firmly together by steel hoops, usually of round rod. Pressure expected inside pipe determines size and spacing of hoops.

Laying. Uniformity of grades and gentle vertical curves are necessary for a well-laid line. The most important consideration, next to quality of pipe, is the nature of the backfilling. Earth containing vegetable mold, roots, grass, sod or other unstable material should never be put within 2 or 3 ft. of the pipe, and preferably not into the trench. Clean selected earth, clay, sand or gravel make the best backfilling. Backfill should be thoroughly rammed around the pipe and for the whole depth of trench. Never allow loose rock to be put into the trench until the pipe has been well covered, generally to a depth of 2 ft., with well compacted earth. Supply line should have air valves at all summits, and blow-offs at important depressions. (See p. 430.)

Wood versus Steel Pipe. In stave pipe, the rods are for strength only; and while steel pipe sometimes has to be abandoned when only 5 per cent. of its strength is destroyed by corrosion, stave pipe would continue tight, and the bands would not be stressed beyond limit until 75 per cent. of the cross-section had rusted away at one or more places; bands, being round, are better able to resist corrosion than thin sheets. Steel pipe is subject to pitting on the inside and rust outside working toward each other. Bands or wires are always of a greater diam. than the thickness of a steel pipe designed for the same service; the wire is double galvanized, and in addition is given a heavy dip of tar or asphalt. It is claimed that machine-banded wood pipe can be transported and laid at about one-tenth cost of steel pipe; is cheaply maintained, and can carry 10 to 35 per cent. more water, with equal diam. (See also p. 571.)



a *b*

FIG. 197.—*a*, Malleable iron lug for round bands with straight pull on pipe lines, tanks, etc., Racine Tank Lug Co., Racine, Wis.; *b*, malleable shoe for wood pipe, Marion Malleable Iron Works, Marion, Ind.

Table 95. Economic Proportions for Continuous Wood Stave Pipe Design
(A. L. Adams, T. A. S. C. E., Vol. 41, 1899, p. 76)

Nominal diam. of pipe, in.	Stock sizes for staves, in.	Thickness of finished staves, in.	Economic sizes of bands, in.	Working stress in band, lb.	Factor of safety in band
			Oval		
10	$1\frac{1}{2} \times 4$	$1\frac{1}{16}$	$\frac{5}{16} \times \frac{7}{16}$	1250	5.3
12	$1\frac{1}{2} \times 4$	$1\frac{1}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1470	4.5
14	$1\frac{1}{2} \times 4$	$1\frac{3}{16}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
16	2×6	$1\frac{7}{32}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
18	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
20	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4 0
			Circular		
22	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1510	4.5
24	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1650	4.0
27	2×6	$1\frac{7}{16}$	$\frac{3}{8}$	1650	4.0
30	2×6	$1\frac{1}{2}$	$\frac{1}{2}$	2670	4.5
36	2×6	$1\frac{9}{16}$	$\frac{1}{2}$	2950	4.0
42	2×6	$1\frac{5}{8}$	$\frac{1}{2}$	2950	4.0
48	2×6	$1\frac{11}{16}$	$\frac{1}{2}$	2950	4.0
54	$2\frac{1}{2} \times 8$	$2\frac{1}{8}$	$\frac{5}{8}$	4600	4.0
60	3×8	$2\frac{1}{2}$	$\frac{5}{8}$	4600	4.0
66	3×8	$2\frac{9}{16}$	$\frac{3}{4}$	6600	4.0
72	3×8	$2\frac{5}{8}$	$\frac{3}{4}$	6600	4.0

Durability.* Conclusions as to durability drawn from 10 yrs.' experience, Astoria, Oregon, are: (1) Staves constantly subject to internal water pressure and buried in the ground may be short-lived. (2) Magnitude of water pressure, beyond a moderate head (20 ft.), has little or no influence in preserving wood. (3) Pipe laid above ground not deteriorated to any considerable extent, nor pipe in tunnels from distributing reservoir. (4) Where buried, durability has depended upon soil conditions and depth of cover. (5) When cover exceeded 2 ft., free from vegetable matter, fine-grained and impervious, deterioration was less. (6) Wherever staves have been in contact with loamy earth or earth containing vegetable matter, or covered with porous material, or to depth less than 2 ft., rapid decay has resulted. (7) Decayed staves have been found all around the pipe. (8) Sound staves frequently found contiguous to badly decayed staves. (9) Character of grain, whether slash or edge, has not influenced durability. (10) Bruising staves during erection seems to hasten decay. (11) Decay confined to outside of pipe. (12) Pipe usually has not leaked where thickness of sound wood has remained in excess of $\frac{1}{4}$ in. (13) Malleable cast band fastenings found in good condition. (14) Bands $\frac{7}{16}$ in. diam. considerably rusted save where secured by nut.

This pipe was built of fir; fir decays very rapidly under natural conditions in that vicinity. Importance of keeping wooden pipe full of water at all times cannot be overestimated. Roots of near-by trees and bushes grow to pipe; hasten decay. All forms of construction which remove portion of stave from immediate contact with water should be avoided. At junctions between wood and iron pipes, putting iron inside wood should be avoided. Best make special iron hub to receive end of wood pipe and calk joint with oakum and lead. Much wood pipe has proved satisfactory, durable and economical.—(Arthur L. Adams, in T. A. S. C. E., Vol. 58, 1907.)

*Exhaustive table has recently been compiled by D. C. Henny, Reclamation Record, Aug., 1915.

Arthur L. Adams* cautions against attributing lack of durability to use of fir for staves instead of redwood, as an important redwood pipe line in Southern California, built about same time had not shown much, if any, better results.

Clemens Herschel* stated that in repairing Holyoke wooden dam it had been found that 6-in. timbers which constituted covering of dam, constantly under 20 ft. of water, were rotten from upstream side downstream, until there was no more sound wood left than would form a veneer. It was really curious to see how closely those 6-in. timbers came to being rotten clear through and yet have that veneer on water side preserved, sometimes not more than ⅛ in. thick.

Decay, being a growth of fungus, is communicable, and the soil in a primeval forest, such as traversed by Astoria pipe, containing decaying woody material, may have produced this exceptional result. There are but two conditions under which wood is absolutely decay proof: Absolute dryness, and constant submersion in water; staves of a wood pipe full of water are never in former condition, and can only approximate the latter, and as a consequence are more or less subject to decay, depending upon amount of moisture in outer portion, which in turn is dependent upon pressure. If moisture of staves alone controlled decay, then pipe of wood of any particular kind or quality would last as long in one locality as in another, pressure being the same. This is not true; there are other causes, almost entirely local, which generally cannot be controlled. If local causes cannot be avoided, decay can be arrested somewhat by rendering the moisture condition less favorable by increasing the pressure. Fir decays very rapidly, and pieces of timber similar in form to staves would decay beyond possible use in 2 or 3 yrs. if left subject to natural conditions in that region. In 1904, 24-in. wood pipe was laid at Long Beach, Southern Cal., a portion in ground overgrown by willows (willow grows very rapidly and requires much water); roots covered surface of pipe, and staves deteriorated rapidly; other portion of pipe in perfect condition. This pipe was extension to pipe built in 1900. It was necessary to take apart about 200 ft. of old pipe; staves were so perfect that they were all reused. If air-valves do not allow the air to escape while the pipe is under pressure, it is certain that air accumulates at high points; this accumulation produces condition more favorable for decay.—(Andrew Swickard.)

A 9-ft. pipe of usual construction, 1800 ft. long, supplying water to paper mill, Floriston, Cal., has redwood staves 3⅝ in. thick; mainly above ground, lower half being bedded with earth and stone spalls; under pressure varying from 16 to 35 ft.; built in 1898 and in continuous use ever since, being kept full of water; shows no evidence of decay of staves or corrosion of bands (1907). Boring through pipe showed outside of wood dry, but below surface it was moist. Two lines of 6-ft. pipe supplying Truckee River General Electric Co., built in 1899, are of Oregon pine. Up to 1907 no evidence of decay; supported as pipe first mentioned, upper half being exposed; pressure varies from 6 to 65 ft.—(A. M. Hunt.)*

Oakland Water Co. has wooden stave pipe leading from wells at Alvarado to pumping station, laid in wet, soggy ground; after 11 yrs., examination showed it had all but gone to pieces.—(L. J. Le Conte.)*

* This page and half of p. 362 were abstracted from T. A. S C E., Vol 58, 1907.

T. Chalkley Hatton investigated (1907) wood pipe of irrigation work in West where continuous stave pipe has been used many years, and several municipal and industrial plants where both continuous and machine-made pipe has been in use at least 40 yrs. Continuous stave pipes where bruised either in handling or by cinching bands too tightly, in most instances suffered decay where pressure was less than 50 lbs., also where longitudinal joints were wide at outer surface and contact not good, or at end joints not in close contact, or where a dry crack existed in end of stave before it was built into pipe. In many places where pipes were examined material in contact was at least 50 per cent. vegetable matter, and no evidence of decay was found, although pipes had been in constant use 8 to 14 yrs.; where top of pipe was only partly covered, or had but few inches of dry earth over it, dry rot had extended to depth of $\frac{1}{8}$ to $\frac{1}{4}$ in., but as soon as constant saturation was reached wood was sound. Greatest defects were in end joints, due to saw kerfs not being exactly in position, so that when metal tongue was inserted inner or outer edge of one stave was a little higher or lower than that of its neighbor. Another marked defect was in durability of steel bands. These bands are $\frac{1}{4}$ to $\frac{3}{4}$ in. diam., coated with asphaltum pitch or similar material, and supposed to be free from oxide when placed. Hundreds of bands when delivered had been exposed to weather sufficient time to give them a thick coat of rust. In this condition they were dipped, covering rust but not stopping its deteriorating effects. In many instances threads were badly rusted, not having been coated; and in most instances it would have been dangerous to move the nut, as stripping would have occurred.

Stave pipe of good soft white pine, Douglas fir, cypress or even red cedar, well selected, free from dry or black knots, either air- or kiln-dried, free from cracks, well jointed and secured with steel bands protected from corrosion, will last, under average conditions, as long as cast-iron pipe.

Lynchburg, Va. A conduit built for Lynchburg, Va., 1906, consists of wood stave pipe for pressures up to 83 lbs., steel pipe for higher pressures, and cast-iron pipe for stream crossings, all 30 in. diam. Profile is such as to keep wood pipe always under pressure. Trench is 6 ft. deep. Minimum radius = 200 ft. California redwood staves, 2 × 6 in., 10 to 24 ft. long, surfaces planed to circumference. One radial face of stave has $\frac{1}{8}$-in. bead, $\frac{1}{16}$ in. high, which under pressure is forced into the plane radial face of the adjacent stave. Leakage at ends of staves is prevented by $\frac{1}{8}$-in. steel plates fitted into saw kerfs. Bands are $\frac{1}{2}$-in. mild steel rounds, specified to have ultimate strength of 58,000 to 65,000 lbs. per sq. in. Each band has hemispherical head on one end and cold rolled thread on other so proportioned that rupture will occur in rod; all tests have proved the adequacy of the proportion. Bands are spaced by formula $N = 3.3H$, applicable only to 30-in. pipe under conditions of the specifications, where N is number of bands per 100 ft., H is pressure head in feet. Bands engage malleable cast-iron shoes, so designed that friction on the base will allow slipping around pipe, rather than cutting into wood, during tightening. At least $\frac{1}{8}$ in. of metal in all parts of shoe. While stored, awaiting use, weather caused cracking and checking of ends of staves. Contractor sawed off damaged ends by hand. It was found impossible to do hand-

sawing accurately enough to make joints as tight as desired. Bands and shoes were dipped in hot Pioneer mineral rubber coating, and touched up with Smith's "durable metal coating." Bends were made by springing pipe while loosely banded, driving butt joints tight, when bands were cinched. Heavy rains before backfilling caused pipe to float 5 or 6 ft. in some instances.—(James H. Fuertes. E. R., Sept. 1, 1906, p. 228.)

MACHINE-BANDED WOOD STAVE PIPE

Continuous Stave vs. Machine-Banded Pipe.—Investigation of continuous stave pipe led T. Chalkley Hatton* to the belief that there were too many uncertainties in manufacture; construction must be left to irresponsible, partly unskilled workmen, who in many instances had to work under unfavorable conditions. For these reasons machine-made wood stave pipe is superior, being made and banded by automatic machines run by capable workmen, staves, bands, coating and workmanship being open to inspection under most favorable conditions. Continuous band steel of any width required by pressure and about No. 16 gage, or steel or copper wire of any gage demanded, while free from rust, are run through a bath of warm asphaltum and then wound spirally upon the pipe, carrying a heavy coating of pitch, thus preventing the inner surface of the band from coming in direct contact with the outer surfaces of the staves. As soon as banding is completed, the pipe is run over two rolls moving through a warm bath of asphaltum, thus coating the outside ¼ to ⅜ in. thick. To prevent coating from running, the pipe is immediately rolled in sawdust, which adheres and prevents it being abraded in handling. Extra coat can be applied by repeating the operation. Coating, as soon as cool, becomes tough, and can only be removed by a chisel. 800 lin. ft. of 24-in. pipe have been laid in a day, much of it in 6 to 8 in. of water in trench. Jointing is so simple that inspector, standing beside trench, can tell at a glance whether it has been properly done. Letters from twenty-five municipalities, water companies and industrial establishments where this pipe has been in use from 5 to 46 yrs., in every case testify to its durability.

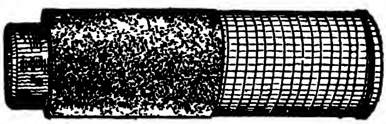


FIG. 198.—Machine-banded wood stave pipe for low pressure. (Protective coating omitted at right to show steel bands. Winding is doubled at end.)

Specifications† (Condensed). *Lumber.* Staves shall be cut from selected white-pine tank plank, free from black knots, checks, shakes or splits, thoroughly seasoned in open air, planed to 1⅜ in. thick. Sections of pipe shall be 3 to 8 ft. long, not more than 10 per cent. under 4 ft. The flat sides of the staves shall be dressed to a smooth surface in true circular lines to radii corresponding to the inner and outer radii of the pipe, and the edges shall be dressed radial. One edge of each stave shall have two grooves cut ⅛ in. deep and ⅛ in. wide at the base; the other edge to have 2 beads $\frac{3}{16}$ in. high and $\frac{3}{16}$ in. wide at base, each groove and bead to be cut ⅜ in. from the inner and outer surface of the stave.

* T. A. S. C. E., Vol. 58, 1907, p. 85.
† Wyckoff Wood Pipe Co., Elmira, N. Y.

Bands. Each section of pipe shall be spirally wound with homogeneous mild steel, having a tensile strength of 58,000 to 65,000 lbs. per sq in., and an elastic limit of about 30,000 lbs., with sufficient tension to bring the edges of staves in close and complete contact throughout. The spacing of bands and gage of steel shall correspond with the pressures designated. When no pressure is designated, the pipe will be wound for 80 lbs. At each end of a section of pipe, bands shall be wound double for one width of band, and the end attached to the staves by 2 or more $\frac{7}{8}$-in. screws. For pipes to withstand 80 lbs. or over, the band shall be further secured to the body of the pipe by 1-in. screws, one screw for every 2 ft. of pipe.

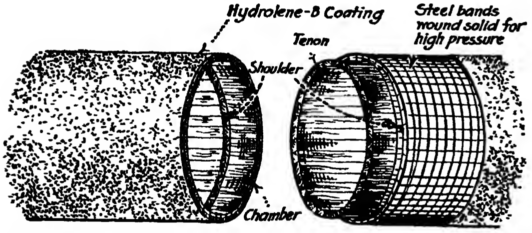


FIG. 199.—Ends of machine-banded wood stave pipe.

Joints. Each section of pipe shall be made with a tenon on one end 4 in. deep with inside shell $\frac{7}{8}$ in. thick, and a chamber on the other end 4 in. deep, with an outside shell, $\frac{7}{8}$ in. thick, this chamber and tenon to be truly and uniformly made so that one can be driven into the other with shoulders butting all around. For pressures of 80 lbs. or over a kerf $\frac{1}{2}$ in. deep and $\frac{1}{8}$ in. wide shall be cut into the face edge of each section of pipe, and a steel band $1\frac{1}{2}$ in. wide of No. 18 Birmingham gage steel, bent to fit into this kerf and riveted together at the ends with flattened rivets, shall be furnished with each section of pipe, the bands to be fitted into the kerfs when the pipe is being laid for the purpose of reinforcing the joints.

Coating Pipe. (See *Coating for Bands*, p. 365.) As the steel is being wound upon the pipe it shall pass through a bath of Special Pipe Coating heated to a temperature of 250° to 300° F., which coating shall completely cover all faces of the band, forming an inner coating between the band and surface of pipe. After winding, each section of pipe shall be thoroughly coated on the outer surface with hot Asphaltene, or asphaltum pitch, put on as thick as the surface will carry, then rolled in a bed of saw-dust which shall adhere to the outer surface of the pipe. When required, the pipe, after the first coating has cooled, shall be given another complete coating of the same material.

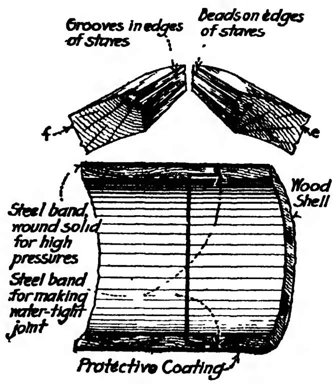


FIG. 200.—Joint between wood pipes, and isometric of two staves. e = grooves in edges of staves, $\frac{1}{8}$ in. deep, $\frac{1}{8}$ in. wide; f = beads on edges of staves, $\frac{3}{16}$ in. high, $\frac{3}{16}$ in. wide at base, $\frac{3}{8}$ in. from outer and inner face. When staves are banded together, the bead, being a little larger than the groove, is squeezed into the latter, making a thoroughly water-tight joint.

Specials. When a change in direction is to be made in the pipe line with a radius less than 200 ft., or where connections are to be made with old or new pipe lines, standard cast-iron specials shall be furnished. Special wrought-iron bands shall be furnished for banding the joints between wood and cast iron. Connection between two wooden pipes shall be made with cast-iron specials having spigot ends only. Connections between wood and cast iron or steel shall have spigot end where the connection is made to

the wood, and bell or flange end, as required, where connection is made with cast iron or steel.

Inspection. Each section of pipe shall be thoroughly inspected before leaving the factory. If, through handling in transportation, the coating is in any way disturbed so as to expose the steel bands, such exposed places shall be recoated alongside the trench with the same material as hereinbefore specified.

Coating for Bands must be durable, hard, tough, perfectly waterproof and strongly adhesive to the metal; it shall show no tendency to flow under summer temperature and not become so brittle as to crack or scale under freezing temperatures. The mixture which will meet these requirements better than any other so far tried, has the following composition: Petrolene, 68.1 per cent.; asphaltene, 27.26 per cent.; total bitumen, 95.36 per cent.

Machine-banded Pipe for Distribution Mains.* In distribution mains cast-iron is better where there is a necessity for tapping these mains for house connections; this exposes the banding of wood pipe to possible damage, both to coating and metal, and thus deterioration begins. Wood pipe should not be used when the normal or constant pressure exceeds 200 lbs. But (1) where there is a supply main connecting the source of supply to the consumer, or to a reservoir, and this main is kept full and constantly under pressure; (2) where there is a supply main from the pump to the reservoir or a consumer, where the pipe is kept full of water; (3) where penstocks are required with water under head, or (4) where water highly charged with acids which tend to destroy iron must be piped, it is claimed that machine-banded wood pipe is quite as durable as cast-iron; more durable than steel; can be laid at less cost; is as cheaply maintained, and can carry more water with equal diameter. One advantage of wood pipe is freedom from electrolysis. Wood pipe, being a non-conductor of heat, can be laid 1 ft. under ground without fear of its being damaged by freezing.

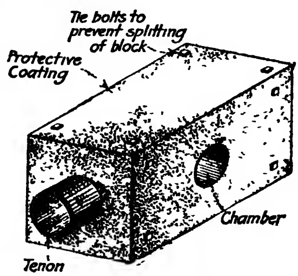


FIG. 201.—Wood elbow for wood pipe, 6 in. and smaller.

Service Connections to Wood Pipe. Bore a hole in middle of a stave with a common wood bit $\frac{1}{32}$ in. smaller than outside diam. of thread on nipple or corporation cock. Connection is then screwed in until its inner end is flush with inside of pipe. When making taps under pressure, bore only until *screw* on bit goes through and water begins to show; connection can then be screwed in far enough to splinter off the thin piece of wood. In this manner taps can be made without drenching the workmen, but a service connection should have cock or valve attached before it is inserted, to hold water while rest of service is being coupled. Blow out thoroughly before connecting to house service to make sure connection is screwed in far enough to get full opening. An extension bit is convenient, as size can be adjusted to a nicety.

Deflections. It is better, if conditions permit, to make a required deflection by the use of a long smooth curve by throwing a slight angle in each joint than by a special fitting. Wire-wound pipe will permit of 2° to 6° deflection

* The following notes are largely from publications of the Wyckoff Wood Pipe Co, Elmira, N Y

in each joint, depending on the size, service head, and skill with which laid. Smaller sizes can be deflected more than the larger, and low-pressure pipe more than high. When vertical or horizontal curves are to be built, include a number of short lengths in the order. After laying a curve, tamp in enough backfilling on the outside of the curve to keep it from buckling and then ram the joints together tight by using the tompion and ram on the last length. Extra care should be taken in backfilling around a curve to see that the tamping is thoroughly done, as there is always a tendency when the pipe is under pressure for it to "work" and blow out on curves. Fittings, particularly bends, should be thoroughly anchored, by placing a strut against the side of the trench, driving in a strong stake and wedging tight against the fitting, or by means of a large stone. Take care that the anchor is not misplaced in backfilling. Plugs should always be held thus, for, while they might hold themselves after becoming soaked, they are apt to blow out when dry.

Capacity. (See also page 571.) Experiments upon old wood stave pipe showed some slight increase in capacity after several years' use. No case is known where a wood pipe kept full of running water has its inner surface rougher after use. "Clean cast-iron pipe would seem to have about 90 per cent. of the capacity of stave pipe, while if seriously tuberculated, a condition which usually prevails after a few years of use, it discharges only about $\frac{3}{4}$ as much as a stave pipe of the same size; a steel pipe discharges when clean from 93 per cent., in the case of a 12-in. pipe, to 68 per cent. in the case of a 6-ft., of the amount which might be expected from stave pipes of the same diam., while if steel pipe is tuberculated to an extent that might readily occur in from 10 to 15 yrs., these discharges may fall to 74 and 54 per cent., respectively."*

Spacing of Bands. From experiments, it was found that a factor of safety of 3 was ample for the steel bands used. Pipes subjected to pressure of 80 lbs. or over and 12 in. in diam. or over, for municipal waterworks, are solidly wound; that is, they are steel pipes with a wooden lining. The band is secured to each end of the pipe by double winding after the end is fastened with 2 or more screws, and on all high-pressure pipes, screws are put through the band into the wood every foot or 8 in. apart, according to diam. and pressure.

Flat Bands vs. Wires. Wood pipe is wound with galvanized iron, steel or copper wire, on order, but is not recommended for the reason that for pipes subjected to an unsteady pressure of 50 lbs. or greater, leaks have been found eventually along longitudinal seams; the higher pressures force the wire into the wood, the lower pressures, following, permit the joint to open. Bands, being $1\frac{1}{2}$ in. wide, cannot sink into the wood, and the excessive pressure must be taken up by the elasticity of the steel; if this pressure is not beyond the elastic limit, the pipe will retain its tightness.

* T. A S. C. E., Vol. 41, 1899, p. 27, Arthur L. Adams.

Table 96. Wood Stave Pipe with Machine-wound Flat Bands

Size of pipe, in.	Pressure, 40 lbs. per sq. in.				Pressure, 80 lbs.			
	Spacing C. to C. for No. 18 bands, in.	Gage of band for solid winding:*	Weight per lin. ft. of pipe, lbs. Steel	Weight per lin. ft. of pipe, lbs. Pipe	Spacing C. to C. for No 18 bands, in.	Gage of band for solid winding:*	Weight per lin. ft. of pipe, lbs. Steel	Weight per lin. ft. of pipe, lbs. Pipe
6	4	.	2.1	10.86	3	....	2.72	11.47
8	$3\frac{3}{8}$	...	2.9	13.65	$2\frac{1}{2}$	....	3.85	14 60
10	3	...	3.75	16.75	2	....	5.55	18.55
12	$2\frac{3}{4}$	...	4.6	19.85	.. .	21	5 55	20.80
14	$2\frac{1}{2}$	.	5.69	23.04	....	19	8 09	25 44
16	$2\frac{1}{4}$	..	7.0	26.62	.	19	8.98	28.60
18	$2\frac{1}{8}$	..	8.0	29.80	....	19	9.90	31 70
20	2	.	9.42	33.39	..	.18	12.55	36.52
24	$1\frac{3}{4}$	..	12.62	40.87	..	17	17.35	45 60
30	. ...	19	17 75	52.50	..	16	22 50	57 25
36	.. ...	18	18 00	59.25	..	14	35 5	76 75
48	.. ..	17	32.2	76.70	.	12	60.4	104 90

Table 96. Wood Stave Pipe with Machine-wound Flat Bands—*Continued*

Size of pipe, in.	Pressure, 120 lbs.			Pressure, 160 lbs.			Pressure, 200 lbs.		
	Gage of band for solid winding:*	Weight per lin. ft. of pipe, lbs. Steel	Weight per lin. ft. of pipe, lbs. Pipe	Gage of band for solid winding:*	Weight per lin. ft. of pipe, lbs. Steel	Weight per lin. ft. of pipe, lbs. Pipe	Gage of band for solid winding*	Weight per lin ft. of pipe, lbs Steel	Weight per lin ft. of pipe, lbs Pipe
6	22	3.0	11 75	21	3 42	12 17	19	4.49	13.24
8	21	4.1	14.85	19	5.39	16 14	18	6 27	17.02
10	19	6.25	19.25	18	7.32	20 32	17	8.68	21.68
12	19	6.68	21.93	17	9.85	25 10	15	12 29	27 54
14	18	9.39	26 74	16	12.45	29 80	15	13.80	31 15
16	17	12.4	32 02	15	15.35	34 97	14	17.75	37 37
18	17	13.65	35 45	14	19.50	41 30	13	22.35	44 15
20	16	16 65	40 62	14	21.3	45 27	12	27 90	50 87
24	14	24 8	53.05	12	32 5	60.75	11	35 85	64 10
30	13	34 5	69 25	11	43 8	78 55	9	53 8	88 55
36	11	51.4	92 65	9	63 3	104 55	7	77 0	121 25
48	8	91.5	136.00	7	100 0	144.50	4	132.0	176 50

Comparative Cost. The cost of laying machine-made pipe is less than for any other pipe for water under pressure; there is no special labor or extra material used in making the joints; the width of excavation is less because no joints are to be made and it is not requisite that men should get around the pipe in laying it; the pipe is light enough to permit being laid without the slow-moving block and fall. Even a 48-in. pipe can be lowered and fitted by 8 men, with 4 rope slings, and 6 of these men can drive the pipe home. In wet trenches the cost of pumping is reduced.

Pipe Laying. (Machine-banded or wire-wound wood pipe.) Care should be taken, no matter what the pressure is to be. Examine each end as the pipe is driven together, and turn the joints to bring any bruised or scratched places to the upper side, as there may be small leaks at these places, which

* Birmingham wire gage.

can be more easily plugged if on top. By the use of tompions and an improvised ram about 5 ft. long, ironed at one end, laying is easy. Pipe 6 in. and larger should be driven together with a ram; smaller sizes can be handled with a maul. Keep the alinement as true as possible, and drive until the shoulders come together at every joint. The pipe is usually driven from the coupling end of wood-coupling pipe, or the female end of inserted joint pipe. If necessary to cut the pipe, mark the place and drive at least two strands of wire together far enough back on each side to allow the required length of tenon. Staple the wire securely before cutting. If necessary to install fittings in an inserted joint line, cut a length of pipe in the manner described at some point in the body, which will save the trouble of making a new tenon on the end, and the ends made by cutting can be made to fit perfectly into the bell of the fitting by rasping. Tenons on wood-sleeved pipe fit the bells of the fittings, but this pipe is best handled as above, when the fitting comes midway in a length. In taking outlets from a line already laid, a saddle can be used; attach it to the pipe by cutting the wire after stapling, and stripping for a space the width of a saddle. Then fit the saddle to the pipe, and mark the point at which the opening is to be cut. After the opening is cut, place a rubber gasket under the saddle and attach with U-bolts or bands.

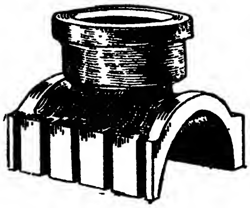


FIG. 202.—Saddle or split tee.

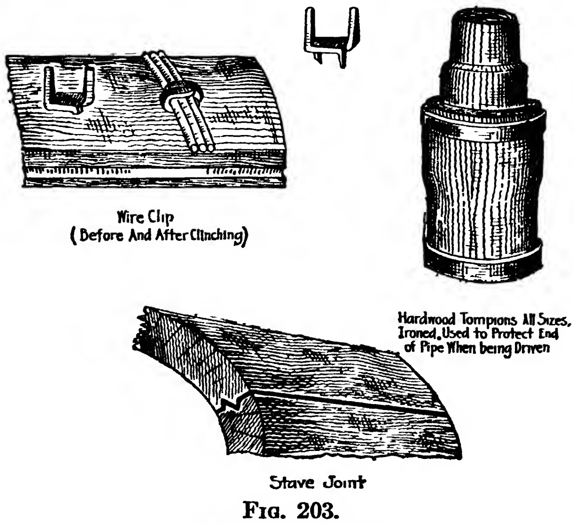


FIG. 203.
(Pacific Coast Pipe Co.)

Head for Laying is conical frame of angle iron with a wooden end at apex for ramming against; the base of the cone is inserted in the chamber of the last pipe laid and struck with an 8-in. by 8-in. by 10-ft. pine timber with three round handles passing through by which 6 men drive the pipe home. For pipes less than 30 in. in diam., 3 thicknesses of 2-in. plank, or less, are used as a

head (also called plug), the inside of the planks having a projection which fits the chamber of the pipe, the head being made square on outside and fitting over the whole end of the pipe, the hammer striking this plug direct. In large or small pipes it takes but a few strokes of the hammer to drive them home.

Eastern Manufacturing Co., Elmira, N. Y., manufactures "Elmira Wood Pipe" 6 in. to 60 in. in diam., and to withstand pressures up to 200 lbs. Pipe 6 in. and over in diam. is made from staves of uniform width and thickness, planed on both sides to a cylinder corresponding to diam., edges radial, beads left on one edge, and corresponding grooves on the other; the thickness corresponds to the pressure, maximum $2\frac{7}{8}$ in. The pipe comes in lengths 8 ft. and under; tenon and socket joints are 4 in. deep.

Strengthened Round Wood Pipe is smoothly bored from white pine logs, free from sap, and is banded with steel bands spirally wound by machine. The pipe is then coated outside with asphaltum pitch and rolled in sawdust. It is perfectly air- and water-tight. Sizes: $1\frac{1}{4}$, $1\frac{1}{2}$, 2, 3, 4, 5 and 6 in. It is tested for 43, 86 and 173 lbs. pressure, as required.

Michigan Pipe Co., Bay City, Mich., manufactures "Improved" wood stave pipe, in sections about 8 ft. long, of well-seasoned white pine and Michigan tamarack staves machined on the sides, forming a double tongue and groove; the inner and outer edges are dressed to conform to an exact curve, corresponding to the diam. of the pipe. The pipe is spirally banded with galvanized steel wire or hoop iron, from end to end, under heavy tension. One end has a mortise, the other, a tenon. Sizes up to 42 in. Each length is tested to 200 lbs. per sq. in., after which the outer surface is heavily coated. Hard maple, beech and birch pipe for flushing culm and similar uses is manufactured like water pipe, from selected dry hard timber, with an extra thick shell and tenon tested to any pressure up to 200 lbs. per sq. in. For pressures of 80 lbs. and lower, wood tees, crosses, ells and bends are manufactured which are especially adapted for mine and acid use. Where long sweeps can be made, this company furnishes, without additional charge, special "curved lengths," so headed as to drive up shoulder to shoulder and form an angle; 90° bends can be formed without cast-iron bends, on about 30 ft. radius.

Pacific Coast Pipe Co. makes wire-wound wood pipe in convenient lengths, generally 8, 10, 12, 14 and 16 ft. Each joint is a butt joint, enclosed in a sleeve made to a high-pressure fit. The internal diam. is maintained uniform. Regular sizes: 2, 3, 4, 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, and 24 in. Staves are of selected fir, milled with precision. The wire is wound under heavy tension (usually $\frac{2}{3}$ tensile strength of wire) by a regulated mechanical device; the size and spacing of the wire are determined by the pressure to which pipe will be subjected. Wire-wound pipe is used for heads up to 350 ft. Care is required in laying, fitting and driving the joints snugly, shoulder to shoulder, without abusing the pipe; and above all, in properly bedding and tamping the earth around the pipe, and in the entire trench. The character of the earth backfill has much to do with the life of the pipe; if filled with sod, roots or other material which will rot, the line can never last so long as when the material is selected; this applies particularly to that portion of the backfill which comes in close contact with the pipe. Staves not milled to mathematically

true radial lines will never give other than a leaky pipe. A pipe not carefully machine wrapped with the full number of turns required by the pressure, and having a liberal factor of safety (never less than 4) is unsatisfactory. All spaces are calculated on an ultimate strength of wire of 60,000 lbs. per sq. in.

Wire. Fig. 204 shows exact sizes of wire ordinarily used on machine wire-wound wood stave pipe by Pacific Coast Pipe Co.

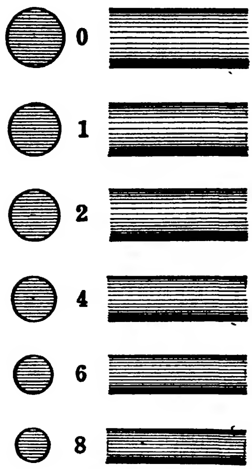


FIG. 204.

Gage*	Diam., in.	Area, sq. in.	Strength	
			Relative	Breaking at 60,000 lbs. per sq. in.
0	0.307	0.074	3.60	4440
1	0.283	0.063	3.06	3774
2	0.263	0.054	2.64	3258
4	0.225	0.040	1.93	2388
6	0.192	0.029	1.4	1734
8	0.162	0.021	1.0	1236

* American Steel & Wire Co. Same as Roebling, and Washburn and Moen.

Coating. The pipe is coated on the outer surface only with tar and asphalt mixed; tenon ends are protected from the coating, so as to secure a tight fit at the joints. The coating is very heavy —much more than would adhere to a metal pipe and remain—thus the wood as well as the banding is well protected. After being coated, the pipe is rolled down an incline covered with sawdust, which adheres to the coating and makes a skin not easily abraded.

Couplings. The inserted joint coupling is made by reaming the inside of the pipe on one end, 2½ to 4 in., according to size of pipe, and on the other end trimming the outside correspondingly. The machine work having been done accurately, the two ends are driven together tightly; water passing through the pipe causes it to swell, thus making an excellent coupling for low-pressure pipe of not too large diam. The wood *sleeve* coupling is made in same way as the pipe, but of greater diam., and the wire banding is spaced much closer. Tenons are formed on the outside of the pipe ends to correspond with the inside diam. of the coupling and slightly tapering toward the ends. The pipe is driven into the sleeve, and the swelling of the wood, after the water is turned into the pipe, makes a strong, tight coupling for any size pipe. Sleeve couplings for 14-in. pipe and over are fitted with individual bands of a size and number to suit the pressure, provided with shoe, nut and washer, and tightened with an ordinary wrench. These couplings may be taken off in pieces and any length of pipe removed and replaced without disturbing the remainder of the line or destroying the coupling.

Cast-iron Fittings. Cast-iron collars can be furnished tapering out to the end from the center of the inside, into which the ends of the pipe can be driven. Machine-banded pipe can be fitted to standard cast-iron fittings having bell or hub ends, but makers furnish lighter fittings, amply strong, and smooth finished in the bell, at less cost, because lighter, and which save labor in fitting the pipe.

Fittings can be machine-threaded to connect with screwed pipe at the additional cost of machine work, or bells made to fit any standard pipe.

Distribution System. Wire-wound pipe can be manufactured to withstand safely 500-ft. head; a large pipe is in successful use under 280 lbs. pressure, equal to 650-ft. head. Taps or service connections require simply the boring of a hole with a bit $\frac{1}{32}$ in. smaller than the outside diam. of the iron service pipe, which is then screwed into the hole; the swelling of the wood, with water in the pipe, holds the tap like a vise, so that a wrench will be required to remove it. After a pipe is laid and it is desired to install a tee without cutting the pipe in two, a saddle which is clamped to the pipe can be furnished with hub end, screw-threaded connection or flange connection.

Weight, Pounds per Linear Ft. of Steel Banding for Machine-made Wood Pipe

(Eastern Mfg. Co)

No. gage	Width, inches		No gage	Width, inches		No gage	Width, inches			No. gage	Width, inches
	¾	1		1¼	1½		1¾	2	2½		3
	lb.	lb.		lb	lb.		lb.	lb	lb		lb.
6	.5078	.6770	6	.846	1.016	4	1.367	1 562	1 953	4	2.344
8	.4296	.5729	8	.716	859	6	1.185	1 354	1.693	5	2.188
10	.3515	.4687	10	.586	.703	8	1.003	1 146	1 432	6	2 031
12	.2734	3645	11	521	.625	9	.914	1 042	1 302	7	1 875
14	.1953	2604	12	456	547	10	.820	938	1 172	8	1 719
16	.1562	.2083	13	391	.469	11	.729	833	1 042	9	1.563
17	.1367	.1822	14	326	.391	12	.638	729	.911	10	1 406
18	.1171	.1562	15	293	352	13	547	.625	.781	11	1 250
19	1074	.1432	16	260	313	14	456	.521	.651	12	1 094
20	.0976	.1302	17	229	273	15	410	.469	58	13	938
21	.0877	.1169	18	195	234	16	365	417	.521	14	781
22	.0781	.1041	19	179	215	17	319	365			
23	.0705	0939	20	163	195	18	273	313			

Velocities of Flow of Water in Machine-made Wood Pipe*

(Feet per second)

Fall or slope		Diameter of pipe, inches												
1 in	Per 100	2	3	4	6	8	10	12	14	16	18	20	22	24
20	5	4 5	5 8	7 1	9 3									
30	3 33	3 7	4.7	5 8	7 6	8 8								
40	2.50	3 2	4.1	5 0	6 6	7 6	8 7	9 5						
50	2 00	2 8	3 7	4 5	5 9	6 8	7.7	8 5	9 2	9 8				
60	1 66	2 6	3 3	4 1	5 4	6 2	7 0	7 7	8 4	8 9	9 5			
80	1 25	2 2	2 9	3 5	4 7	5 4	6 1	6 7	7 3	7 7	8 2	8 7	9 2	9 8
100	1 00	2 0	2 6	3 2	4 1	4 8	5 5	6 0	6 5	6 9	7 3	7 8	8 2	8 7
150	0 67	.	2.1	2 6	3 4	4 0	4 4	4 9	5 3	5 7	6 0	6 3	6 7	7 1
200	0 50	. .		2 2	2 9	3 4	3.9	4 2	4 6	4 9	5 2	5 5	5 8	6 2
300	0 33	...	.	.	2 4	2 8	3 2	3 5	3 8	4 0	4 2	4 5	4 7	5 0
400	0 25				2 1	2 4	2 7	3 0	3 3	3 5	3 7	3 9	4 1	4 4
500	0 20					2 1	2 4	2 7	2 9	3 1	3 3	3 5	3 7	3 9
1000	0 10				..		.....	1.9	2.0	2 2	2.3	2 5	2 6	2 8

*Pacific Coast Pipe Co.

Noteworthy wood pipe lines: Astoria City, Ore.; West Los Angeles; Ogden, Utah; Floriston, Cal. (all in T. A. S. C. E., Vol. 58, 1907); Bear Valley, Cal., T. A. S. C. E., Vol. 33, 1895; Lynchburg, Va., E. R., Vol. 54, 1906.

CHAPTER XVII

CONCRETE AND REINFORCED CONCRETE PIPES

Concrete Pipe. For heads under 15 ft. best mixture for pipes up to 20 in. diam. is cement 1 : 4 sand and gravel. For larger pipe, use 1 : 3 mix. To all batches add hydrated lime, 5 per cent. by weight of cement. Cast on end. Every inch of circumference should be carefully tamped; do not displace core when so doing; otherwise pipe will vary in thickness. Pipes even of best materials and workmanship may be ruined if not properly cured. Curing should take place in open air unless there is danger of freezing. After pipe has set for about 30 min. sprinkle by hand with fine sprinkler. Drench after 2 or 3 hrs., and wet often enough after that to keep from drying out, for 1 week. After 7 days' curing, coat the pipes with a thin mixture of cement and water, pipes 4 to 12-in. diam. being dipped. All pipes should cure for 3 weeks before being placed in the trench. Cement at pipe joints should be allowed to set 1 hr. before refilling the trench; use only earth free from stones and tamp lightly around joint. When cover of 1 ft. above joint has been put in, ordinary refilling may be used; 36 hrs., or preferably 48, should be given joints to set before turning water into the pipes. (J. A. Griffin, Los Angeles, E. R., May 6, 1911, p. 505.)

Lock Joint Pipe. Concrete pipes reinforced with wire fabric, cast in lengths along the trench or in a yard, according to Meriwether system (patented) are furnished by Lock Joint Pipe Co., N. Y. City. For Hamilton, Ont., 24-, 27-, 30-, 33-, 36-, 42-, and 48-in. pipes were supplied, and for Havana sewer system, up to 84-in. Steel molds for casting bell-and-spigot reinforced concrete pipes in standard lengths (*e.g.*, 48 in. diam., 8 ft. long) are made by Blaw Collapsible Steel Centering Co., Pittsburg.

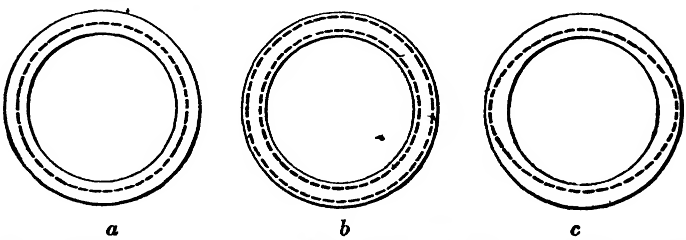


FIG. 205.—Reinforced concrete pipe. Three positions for mesh reinforcement: *a*, One layer to resist internal pressure; *b*, two layers for great depths or high internal pressure; *c*, one layer to resist external pressure.

Various kinds of metal are used for reinforcing: triangular mesh, expanded metal, bars. Reinforcement extends the length of the section and projects both into the bell end and out of spigot end several inches. Spigot is shorter than bell; therefore when two sections of the pipe are placed together the reinforcing metals overlap in an internal recess. This recess is filled with cement

mortar, locking the sections together and sealing the joint at one operation. Filling is accomplished through a hole at top of bell in smaller pipes. Joints in 36 in. and larger pipes are made from the interior by forcing grout behind a shield with a cement gun. Pipes are cast in 4-ft. lengths. Thickness varies from 3 in. for 24-in. pipe to 8 in. for 84-in. pipe. Length of internal recess is $1\frac{3}{8}$ in. to $1\frac{7}{8}$ in.

Reinforced Concrete Pipe Co., Jackson, Mich., builds pipe along the route in steel forms, which consist of bottom ring, inner form, outer form and collar; bottom ring is cast-iron and shaped to form female end, and has small shoulder on inner and outer edge to receive inner and outer forms. Pipe is cast on end. At equal intervals around this plate are holes to receive longitudinal reinforcing bars. Inner and outer forms are interlocking steel plates formed into arc of circle; upper ends are held by spacing clips that also hold longitudinal reinforcing bars in position.

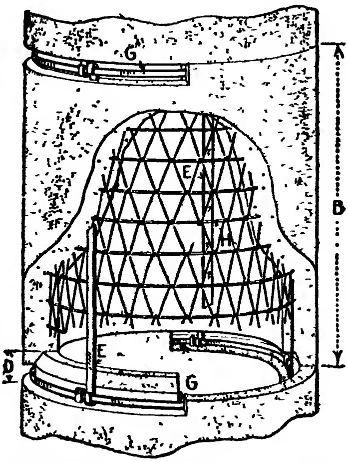


FIG. 206.—Standard reinforced concrete pipe. See Table 97, page 374.

As concrete is being rammed into form, circular reinforcing bands are inserted by removing spacing clips and dropping bands into soft concrete. When mold is filled to top of outer form, about 3 in. shorter than inner form, cast-iron collar is put on and space inside filled; collar makes male end of pipe.

Concrete is allowed to set 8 hrs., when inner form may be removed, and in 24 hrs. pipe can stand without the outer form. Usually 10 days must elapse before pipe can be turned over and the bottom ring removed, and should not be lifted until about 14 days old. Longitudinal reinforcing bars, E, are bent into hook at each end and project about $1\frac{1}{2}$ in. beyond concrete. When pipe is laid, hooks lap and curved bar, G, can be passed through, locking two joints together; space about interlocking bar is then grouted.

Experience. From tests and experience the following general conclusions may be drawn: (1) Pipes should be designed (*a*) so that when reinforcement and concrete are considered as acting together stress in concrete will not exceed safe tensile strength, or (*b*) so that longitudinal joints will be formed at

Table 97. Details of Standard Reinforced Concrete Pipe,* Fig. 206

Dimensions of pipe				Size of reinforcement				
Diam., in.	Length B, ft	Thickness of wall, in.	Length of flange D, in.	*Bars E, in.	Bands, in.	Tie bands G, in.	A. S. & W. Co. triangle wire mesh, style No †	No. longitudinal bars per section of pipe
24	3	$2\frac{1}{2}$	3	$\frac{1}{8} \times \frac{1}{2}$	$\frac{1}{16} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{3}{8}$	6S	3
27	3	3	$3\frac{1}{8}$	$\frac{1}{8} \times \frac{1}{2}$	$\frac{1}{16} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{1}{2}$	6S	3
30	3	$3\frac{1}{2}$	$3\frac{3}{4}$	$\frac{1}{8} \times \frac{5}{8}$	$\frac{1}{8} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{5}{8}$	5S	3
33	3	$3\frac{3}{4}$	4	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	5S	3
36	3	4	$4\frac{1}{8}$	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	5S	3
39	3	$4\frac{1}{4}$	$4\frac{1}{4}$	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	5S	5
42	3	$4\frac{1}{2}$	$4\frac{3}{8}$	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times 1\frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	26S	5
48	3	5	$4\frac{3}{4}$	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times 1\frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	26S	5
54	3	$5\frac{1}{2}$	$4\frac{7}{8}$	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times 1\frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	25S	5
60	3	6	5	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times 2$	$\frac{1}{8} \times 1$	25S	5
66	3	$6\frac{1}{2}$	$5\frac{1}{8}$	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times 2$	$\frac{1}{8} \times 1$	25S	7
72	3	7	$5\frac{1}{4}$	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times 2\frac{1}{4}$	$\frac{1}{8} \times 1$	26D	7
78	5	8	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	26D	5
84	5	8	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	26D	5
90	5	$8\frac{1}{2}$	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	25D	8
96	5	$8\frac{1}{2}$	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	25D	8
102	5	9	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	24D	8
108	5	9	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	24D	8
114	5	$9\frac{1}{2}$	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	23D	8
120	5	10	6	$\frac{5}{16} \times 1\frac{1}{4}$	....	$\frac{3}{16} \times 1$	23D	8

* Reinforced Concrete Pipe Co., Jackson, Mich.
† Sometimes used instead of rod reinforcement, as shown in Fig 206. S = single, D = double.

definite places, to be subsequently grouted while pipe is under slightly excessive stress, but such grouting is not ordinarily practicable. For heads of 60 ft. and less, there seems to be no reason why cracks cannot be prevented by proper thickness of concrete and ratio of steel. (2) Porous spots should be grouted and pointed from inside, and entire interior surface covered with well-brushed coat of neat cement grout. (3) Finer aggregate should contain high percentage of fine material, about 80 per cent. passing No. 16 sieve, and about 33 per cent. No. 50 sieve; natural sand preferred to stone screenings. (4) Economic limit to head to be carried by reinforced concrete pipe is probably 100 to 125 ft. (5) Leakage in properly constructed pipe is no greater than in cast-iron pipes of same size, and, besides, continually decreases. (6) A well-made pipe may be considered practically a permanent structure. (7) Higher coefficient of discharge can be used than with steel pipe. (8) Under reasonable management, cost should be less than for steel pipes up to 100 to 125-ft. head; depends largely upon local conditions as to cost of concrete materials and labor, as compared to transportation from shop and price for steel pipe. Most of these remarks apply especially to large pipes or conduits.

Reinforced Concrete Aqueduct. F. F. Moore in Jl. Ass'n. Engg. Socs., Vol. 47, July, 1911, outlines designing methods for reinforced portions of Catskill aqueduct. Maximum head on center of pipe is 50 ft. Diameters vary from 7.75 to 17 ft. For large diameters, not far below hydraulic gradient, loads are not uniform around the circumference. Internal load results from hydro-

static head, generally about 2 to 3 times the diameter of pipe, giving greater pressure at bottom than at top. External load comprises weight of masonry, refill and embankment. Non-uniform loading induces in ribs, besides direct stresses, flexural stresses of important magnitude. Experience with experimental section of aqueduct, supported by experiments conducted by U. S. Reclamation Service, led to adoption of 200 lbs. per sq. in. for maximum working tensile stress in concrete (concrete and steel acting together), and 6000 lbs.

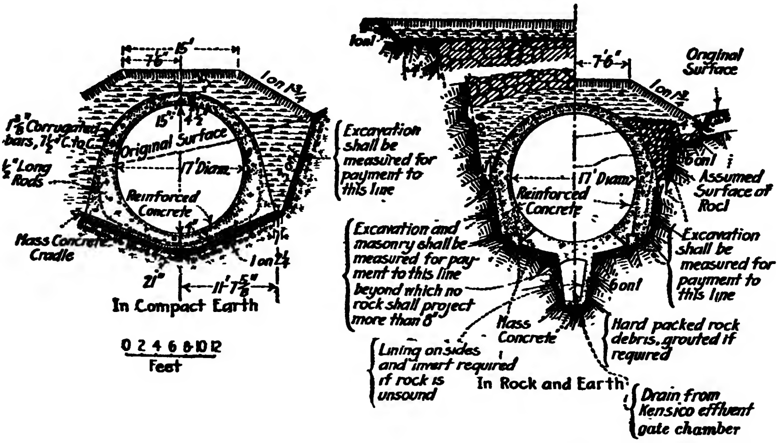


FIG. 207.—Reinforced concrete aqueduct.* Types in open cut. (Kensico effluent, Catskill aqueduct)

per sq. in. in steel when concrete takes no tension as at an accidental joint. Sections were investigated for controlling load conditions at all critical planes with reference to both stresses; in general, design of steel to take all stress met condition imposed for concrete and steel acting together.

Designed sections differ somewhat to suit local conditions (see Fig. 207), but consist essentially of a masonry ring with transverse steel rings embedded near inner and outer surfaces. As a guard against corrosion from percolating waters, and to allow for inaccuracies in placing, steel is everywhere covered with nominal thickness of not less than 4 in. of concrete. The masonry is gradually thickened from top to horizontal diam., and the side-bottom is widened to provide adequate foundation. In rock cuts, affording firm side support and anchorage, a thinner rib with no reinforcement is used below horizontal diameter. Trial sections, perfected sufficiently to answer the purpose, were used to determine economic shape of masonry section, and relation between concrete and steel areas. This involved investigation of several possible loadings to discover critical position. Stress determinations were largely graphical. Resultant force lines were drawn in usual way; that force line which fixes position of center of internal stress in materials being determined by Castigliano's theorem of least work.† This theorem recognizes the

* Notable reinforced concrete aqueducts are: Grenoble, France (E. R., Mar. 7, 1903); Jersey City (E. R., Jan. 16, 1904); Cedar Grove, N. J. (E. R., Dec. 12, 1903); Huesca, Spain (E. R., Dec. 3, 1910); Pinto and Cottonwood, Ari. (T. A. S. C. E., Vol. 60, 1908); Forest Boise project, Ida. (E. N., Aug. 8, 1912); Saugus, Los Angeles (Eng. Contr., July 3, 1912).

† Burr, "Elasticity and Resistance of Materials of Engineering," 7th edition, p. 788, 1915, Wiley.

fact that distribution of stress in a stable, elastic rib supporting external forces will be such that aggregate work performed by internal forces, or stresses, is minimum. In aqueduct sections external forces were generally symmetrical about vertical axis, so that direction of resultant forces at middle of crown and invert could be taken as horizontal. True line had to be sought by laborious "cut-and-try" process, although an experienced man could get results in three or four trials. Actual work done need not be determined, as we are interested only in differentials; certain common terms and factors can be eliminated. Resultant force line corresponding to least work done by internal stresses is indicated by line giving least summation of the values of $\frac{M^2}{I} + \frac{P^2}{A}$.

A = area of concrete section of assumed radial plane,
P = component of resultant force normal to assumed radial plane,
n = distance from point of application of P to center of radial plane,
h = length of radial plane,
I = moment of inertia of concrete section, about its center of gravity, $= h^3$
M = bending moment of force, P, $= P \times n$

In above expression, $\frac{M^2}{I}$ refers to influence of P in producing flexure; second, to direct stress influence. Length of aqueduct 12 in. long was considered; all quantities taken in inches and pounds. In cases studied P was a tension force, but a condition of loading is conceivable which would make P a compressive force on parts of rib.

Obvious value of I in above expression is moment of inertia of reinforced section with respect to its neutral axis, but use of I for plain concrete section will answer our purpose, since reinforcement consists of bars symmetrically placed with respect to an axis through center of concrete section, so that this axis coincides with neutral axis. This substitution is based on the assumption, reasonable within limits set for working stresses, that modulus of elasticity of both concrete and steel are respectively constant and of same value for both tension and compression. I of reinforced section is larger than I for plain section by a quantity which is constant in each joint for all values of M; as we are interested only in differentials, use I for plain section. Substituting for M, I and A for section 12 in. long, criterion may be re-written $\frac{P^2n^2}{h^3} + \frac{P^2}{h}$.

Having thus determined true resultant force line and force polygon, stresses are readily obtainable. Condition assuming no tension in concrete gives rise to 3 cases: (1) reinforcement of both sides of rib in tension and no compression in concrete. This obtains when resultant external force intersects assumed radial plane near its center; is solved by resolving component of resultant into 2 parallel forces acting at centers of steel bars; (2) when part of concrete, including one reinforcing bar, is in compression, while other bar is in tension. (3) Reinforcement of both sides in tension, and some compression in concrete. For cases 2 and 3, cubical equations were derived containing 2 unknown quantities: area of steel and distance from neutral axis to most compressed

fiber of concrete. Solved by trial, generally by assuming a commercial size of bar.

Forms. Solid outside forms should not be used for large conduit built in place, as access is difficult and it is practically impossible to obtain smooth, dense inside surface, especially in invert. The ribs should be made of two continuous halves, composed of an I-beam or double channels, bolted together at the top, and held to position by longitudinal separators. At the bottom the ribs should be held by longitudinal plate about 1 ft. high, which would act as the first course of lagging. Lagging should be of wood, about 8 in. wide, and be provided with a 2-in. × $\frac{1}{8}$-in. steel plate, screwed on to the outside face of the board, on its lower edge, and projecting over the edge about 1 in. These are intended to make a tight joint. If calking is required, cotton packing may be used. Advantage of plate on the lower edge of the lagging is that there is no offset in the upper edge of the board to become filled with concrete, but just the square edge, which can be easily brushed. Lagging should rest against the ribs, and be held in position by a hardwood or metal button.

Reinforced Concrete Pipe on Umatilla Project, Oregon, U. S. Reclamation Service. Line of 46-in. pipe, 4700 ft. long, maximum head, 55 ft. Pipes cast near site. Test sections warranted the following conclusions which have been borne out by construction and operation: (*a*) Circumferential reinforcing steel can be stressed to about 12,500 lbs. per sq. in. without causing appreciable leakage; (*b*) cement joint between pipes could resist 30 lbs. per sq. in.; (*c*) 3-in. shell would amply resist internal pressure and ordinary handling; (*d*) a coat of plaster $\frac{1}{2}$ in. thick well applied would successfully resist percolation under pressure of 100 ft. Collapsible steel forms were used. Collars were 3 in. wide and 3 in. thick. Expanded metal proved poor reinforcement because of a tendency to crack along the plane of reinforcing; a $\frac{1}{2}$-in. mortar inside coating was found highly expensive and abandoned; 1 : 2.3 : 3 mix used (gravel). Reinforcement, $\frac{5}{16}$-in. mild steel wire, wound on a drum into a helical coil, spaced according to the head, to give unit stress of 12,000 lbs.

Tests on a 46-in. pipe under 110-ft. head showed that pipe 3 in. thick would stand 50 lbs. per sq. in. without excessive leakage. Mix, 1 : 1.44 : 2, gravel; reinforcement, a double coil of $\frac{5}{16}$-in. wire, with 1.25-in. spacing.—(H. D. Newell.)

Pipe at Roosevelt, Ariz. U. S. Reclamation Service. Tests on 2400 ft. of 5-ft. 3-in. pipe at Roosevelt, Ariz. (1905), under 35 to 40-ft. head, indicated a loss of 0.05 per cent. of the water passing through, after first filling. In a few months, the leakage diminished to 180 gals. per hr. (0.0094 mgd. per mile). The leakage on the first test was largely due to several large cracks in the first stretch constructed. Teichman's alligator machine was used; is said to be cheaper than forms and gives a smooth pipe, practically continuous (T. A. S. C. E., Vol. 60, 1908). No plaster coat was used; it is not recommended. Wet concrete of small aggregates and plenty of sand, thoroughly worked, produced a pipe not only non-porous, but hardly permeable. Mix was 1:2:4 to 1:2.5:4.5, depending on the gravel, all of which was fine, in no case exceeding 1 in. The pipe was 6 in. thick but unless the pipe is constructed under conditions where it will be continuously damp it should be made as thin

as practicable; about 4 in. is the minimum thickness obtainable with the Teichman machine. At Roosevelt, one pipe, built continuously, leaked 60 gals. per min.; companion pipe, built under much worse conditions with joints between days' work every 40 ft., leaked about 400 gals. per min. Although leaky pipe is 2 in. thinner, leakage seemed to occur only at ends of days' work. Due to removing forms too soon, incipient separation in plane of reinforcement rings between concrete on either side of reinforcement rings occurred in some places.

Construction Methods. In a 5-ft. circular pipe of U. S. Reclamation Service, for 65-ft. head, continuous longitudinal reinforcement amounts to 0.2 per cent. Pipe was built in lengths of days' work (63 ft. maximum) stopped at a vertical plane by cement bags filled with sand; before continuing work next day, projecting fins were removed with a hammer. In one month (of comparatively constant temperature) in the fall of 1907, 550 ft. were placed without longitudinal cracks. Concrete was covered with earth as soon as possible and kept wet to prevent drying-out. Bad places in concrete were cut out immediately on discovery, washed with neat cement grout, and filled with concrete. Slow-setting cement was used; forms were left up 48 hrs. Concrete mix, 1 : 2.4 : 3.6. Used 23 to 24 per cent. water. Field welds of the reinforcement showed an efficiency of about 50 per cent. of ultimate strength, and 100 per cent. of elastic limit; the latter should be the critical point for such work.—(W. W. Patch, U. S. Reclamation Service, Dec. 28, 1907.)

Tuberculation. In Jersey City reinforced concrete conduit tuberculation of steel has occurred where too near the surface of the concrete, like tuberculation on inner surfaces of steel pipe. Same phenomenon has been observed in some other pipes.

Cement Pipe. Paterson, N. J., laid cement pipe (cement-lined sheet-iron covered with cement) 4-in. to 12-in. diam., 1855 to 1877. Sheet-iron sleeves at joints. In 1897 about 28 miles in use; in 1908, 7 miles. Pressure 20 to 70 lbs. On removal large percentage of pipe was in good condition. Metal found to be rusted out in many places. Workmen on other subterranean work were continually breaking into cement pipe.

Cement-lined Pipe in Use; Number of Breaks or Leaks (Paterson, N. J.)
(A. W. Cuddeback, Passaic Water Co.)

Year	Miles	Breaks	Year	Miles	Breaks
1898	26.8	113	1903	15.6	60
1899	24.1	100	1904	15.2	59
1900	21.2	109	1905	10.3	19
1901	18.3	82	1906	7.4	17
1902	16.8	66	1907	7.2	33

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PART IV
DISTRIBUTION OF WATER

CHAPTER XVIII

CAST-IRON PIPE AND SPECIALS

Water is distributed in most American towns and cities through cast-iron pipes. Sept. 10, 1902, New England W. W. Assn. adopted a standard schedule of sizes, shapes and thicknesses and standard specifications for cast-iron pipe and special castings which have been widely adopted. Subsequently other specifications and tables of dimensions and weights were issued by the Am. Soc. for Testing Materials (Nov. 15, 1904), U. S. Cast-iron Pipe & Foundry Co., Am. Cast-iron Pipe Co., Am. Foundrymen's Assn., and Canadian Soc. C. E., which are essentially like those adopted May 2, 1908, by Am. W. W. Assn. and regarded as the manufacturers' standard. The N. E. and Am. W. W. associations publish their standards in pamphlet form, for sale by the secretaries at a nominal charge. Between the two specifications there are but few differences, but the schedules, and the pipes and specials made from

Table 98. Number, Position and Weight of Set-screw Bosses and Bolt Lugs on Special Castings and Valves for Securing Them in Pipe Lines

Diam. of pipe, in.	No. of bosses	No. of pairs of lugs		Position of bosses and lugs. Horizontal diam. taken as 0°				Wt. of 1 boss or 1 pr. of lugs
		Specials	Valves	Str way br.	Y branch	B O. br.	Valves	
4	3			0°&30°		30°&90°		0 8
6	3			0°&30°		30°&90°		0 8
8	3			0°&30°		30°&90°		0 9
10	3			0°&30°		30°&90°		1 1
12	*	4	2	0°&90°		0°&90°	0°	7 7
14	*	4		0°&90°				7 9
16		6	2	0°&60°	30°&90°	0°&60°	0°	8 8
20		6	6	0°&60°	30°&90°		0°&60°	9 2
24		6	6	0°&60°	30°&90°		0°&60°	9 1
30		6	5	0°&60°	30°&90°		†0°&60°	12 8
36		6		0°&60°	30°&90°			13 2
42		8		0°45°90°	22½°&67½°			13.8
48		8		0°45°90°	22½°&67½°			14 2

NOTE. In giving the positions of lugs and bosses above, the smallest angle from either end of the horizontal diam. is given in each case, disregarding the direction of reckoning.

On hydrant branches are 3 bosses, one on diam. parallel to main pipe and the other two at 30° from this diam.

Curves with lugs for bridge connections, etc.—20″ and larger, have 4 pairs of lugs on bells, on 45° diam., and 4 single lugs approximately on 45° diam. 22 in from spigot end

Above table (based on practice of Met. W. W., Boston) or one similar, in accord with local practice, is useful for making pipe layouts, ordering bolts, etc.

* On caps, 3 bosses instead of 4 lugs. In other cases caps are just like branches.

† One pair of lugs on horizontal diam., next to dome is omitted for lack of room.

them, differ materially because of the methods of adjusting diams. to changes of thickness to obtain various classes of pipes for various pressures in service.

In the N. E. schedule there is an inconsistency which should be corrected: for 30-in. and larger pipes and specials the thickness of the bell, or socket, is less than the thickness of the barrel of the pipe, for the heavier classes.

For coating very large special castings modification of the specified requirements is necessary, since there are not large enough heating ovens and dipping vats in existence. A tar or asphalt varnish may be applied with a brush to the cold casting, after thorough cleaning, or somewhat better results may be obtained by locally heating the casting with large gas torches and immediately applying the varnish hot.

FLANGED PIPE AND FITTINGS

1915 U. S. Standard Schedule of Flanged Fittings and Flanges.—Adopted Mar. 20, 1914, by Joint Com. of National Assn. of Master Steam and Hot Water Fitters, Am. Soc. M. E., and Com. of Manufacturers on Standardization of Fittings and Valves. Effective Jan. 1, 1915. Tables 99, 100, and 101 contain committee's schedule (which supersedes all previous schedules); it is accompanied by the following notes:

General.—Size of pipe indicates inside diam. of ports. If flanged fittings for lower working pressure than 125 lbs. are made, they shall conform in all dimensions, except thickness of shell, to this standard and shall have the guaranteed working pressure cast on each fitting. Flanges for these fittings shall be standard dimensions. All standard weight fittings shall be guaranteed for 125 lbs. working pressure and extra heavy fittings for 250 lbs. working pressure, and each fitting shall have some mark cast on it indicating the maker and guaranteed working steam pressure. All extra heavy flanges and fittings shall have a raised surface $\frac{1}{16}$ in. high inside of bolt holes, for gaskets. Thicknesses of flanges and center to face dimensions of fittings include this raised surface. For fittings reducing on the run only a long body pattern shall always be used. Standard weight fittings and flanges shall be plain faced. Steel flanges, fittings and valves are recommended for superheated steam.

Bolts.—Square head bolts with hexagonal nuts are recommended. Holes to be $\frac{1}{8}$ in. larger in diam. than bolts and to straddle center lines.

Elbows.—Standard and extra heavy reducing elbows carry same dimensions center to face as regular elbows of largest straight size. Where long turn fittings are specified, reference is only to elbows made in two center-to-face dimensions, to be known as elbows and long-turn elbows, the latter being used only when so specified. Double-branch elbows, side-outlet elbows and side-outlet tees, whether straight or reducing sizes, carry same dimensions center to face and face to face as regular tees and elbows. Double-branch elbows are not made reducing on the run. Y's are special and made to suit conditions.

Tees, Crosses and Laterals.—Standard and extra heavy tees, crosses and laterals, reducing on run only, carry same dimensions face to face as largest

Table 99. Standard Flanges

(All dimensions in inches)

Pipe size, in	Standard weight. Working pressure, 125 lbs per sq in					Extra heavy. Working pressure, 250 lbs per sq in				
	Flanges		Bolts			Flanges		Bolts		
	Diam. *D*	Thickness *T*	Diam. of bolt circle *C*	Number	Diameter	Diam *D*	Thickness *T*	Diam. of bolt circle *C*	Number	Diameter
1	4	7/16	3	4	7/16	4½	11/16	3¼	4	½
1¼	4½	½	3⅜	4	7/16	5	¾	3¾	4	½
1½	5	9/16	3⅞	4	½	6	13/16	4½	4	⅝
2	6	⅝	4¾	4	⅝	6½	⅞	5	4	⅝
2½	7	11/16	5½	4	⅝	7½	1	5⅞	4	¾
3	7½	¾	6	4	⅝	8¼	1⅛	6⅝	8	¾
3½	8½	13/16	7	4	⅝	9	1 3/16	7¼	8	¾
4	9	15/16	7½	8	⅝	10	1¼	7⅞	8	¾
4½	9¼	15/16	7¾	8	¾	10½	1 5/16	8½	8	¾
5	10	15/16	8½	8	¾	11	1⅜	9¼	8	¾
6	11	1	9½	8	¾	12½	1 7/16	10⅝	12	¾
7	12½	1 1/16	10¾	8	¾	14	1½	11⅞	12	⅞
8	13½	1⅛	11¾	8	¾	15	1⅝	13	12	⅞
9	15	1⅛	13¼	12	¾	16¼	1¾	14	12	1
10	16	1 3/16	14¼	12	⅞	17½	1⅞	15¼	16	1
12	19	1¼	17	12	⅞	20½	2	17¾	16	1⅛
14	21	1⅜	18¾	12	1	23	2⅛	20¼	20	1⅛
15	22¼	1⅜	20	16	1	24½	2 3/16	21½	20	1¼
16	23½	1 7/16	21¼	16	1	25½	2¼	22½	20	1¼
18	25	1 9/16	22¾	16	1⅛	28	2⅜	24¾	24	1¼
20	27½	1 11/16	25	20	1⅛	30½	2½	27	24	1⅜
22	29½	1 13/16	27¼	20	1¼	33	2⅝	29¼	24	1½
24	32	1⅞	29½	20	1¼	36	2¾	32	24	1⅝
26	34¼	2	31¾	24	1¼	38¼	2 13/16	34½	28	1⅝
28	36½	2 1/16	34	28	1¼	40¾	2 15/16	37	28	1⅝
30	38¾	2⅛	36	28	1⅜	43	3	39¼	28	1¾
32	41¾	2¼	38½	28	1½	45¼	3⅛	41½	28	1⅞
34	43¾	2 5/16	40½	32	1½	47½	3¼	43½	28	1⅞
36	46	2⅜	42¾	32	1½	50	3⅜	46	32	1⅞
38	48¾	2⅜	45¼	32	1⅝	52¼	3 7/16	48	32	1⅞
40	50¾	2½	47¼	36	1⅝	54½	3 9/16	50¼	36	1⅞
42	53	2⅝	49½	36	1⅝	57	3 11/16	52¾	36	1⅞
44	55¼	2⅝	51¾	40	1⅝	59¼	3¾	55	36	2
46	57¼	2 11/16	53¾	40	1⅝	61½	3⅞	57¼	40	2
48	59½	2¾	56	44	1⅝	65	4	60¾	40	2
50	61¾	2¾	58¼	44	1¾					
52	64	2⅞	60½	44	1¾					
54	66¼	3	62¾	44	1¾					
56	68¾	3	65	48	1¾					
58	71	3⅛	67¼	48	1¾					
60	73	3⅛	69¼	52	1¾					
62	75¾	3¼	71¾	52	1⅞					
64	78	3⅜	74	52	1⅞					
66	80	3⅜	76	52	1⅞					
68	82¼	3½	78¼	56	1⅞					
70	84½	3½	80½	56	1⅞					
72	86½	3½	82½	60	1⅞					
74	88½	3⅝	84½	60	1⅞					
76	90¾	3⅝	86½	60	1⅞					
78	93	3¾	88⅞	60	2					
80	95¼	3¾	91	60	2					
82	97½	3⅞	93¼	60	2					
84	99¾	3⅞	95½	64	2					
86	102	4	97¾	64	2					
88	104¼	4	100	68	2					
90	106½	4⅛	102¼	68	2¼					
92	108¾	4⅛	104½	68	2¼					
94	111	4¼	106¾	68	2¼					
96	113¼	4¼	108¾	68	2¼					
98	115½	4⅜	110¾	68	2¼					
100	117¾	4½	113	68	2¼					

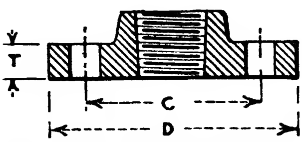
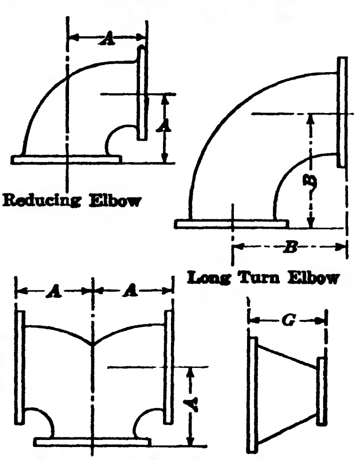


FIG. 208.—Standard flange.

Table 100. Standard Fittings (except short body reducers)
(All dimensions in inches)

Pipe size	B	G	Standard weight. Working pressure, 125 lb. per sq. in. A	C*	D*	E*	Min. metal thickness	Extra heavy. Working pressure, 250 lb. per sq. in. A	C*	D*	E*	Min. metal thickness
1	5	.....	3½	1¾	5¾	1¾	7/16	4	2	6½	2	½
1¼	5½	.....	3¾	2	6¼	1¾	7/16	4¼	2¼	7¼	2¼	½
1½	6	.....	4	2¼	7	2	7/16	4½	2½	8½	2½	½
2	6½	.....	4½	2½	8	2½	7/16	5	3	9	2½	½
2½	7	.....	5	3	9½	2½	7/16	5½	3½	10½	2½	9/16
3	7¾	6	5½	3	10	3	7/16	6	3½	11	3	9/16
3½	8½	6½	6	3½	11½	3	7/16	6½	4	12½	3	9/16
4	9	7	6½	4	12	3	½	7	4½	13½	3	5/8
4½	9¾	7½	7	4	12¾	3	½	7½	4¾	14½	3½	5/8
5	10¼	8	7½	4½	13½	3½	½	8	5	15	3½	11/16
6	11½	9	8	5	14½	3½	9/16	8½	5½	17½	4	¾
7	12¾	10	8½	5¼	16¼	4	5/8	9	6	19	4½	13/16
8	14	11	9	5½	17½	4½	5/8	10	6	20½	5	13/16
9	15¼	11½	10	6	19¼	4½	11/16	10½	6½	22½	5	⅞
10	16½	12	11	6½	20½	5	¾	11½	7	24	5½	15/16
12	19	14	12	7½	24½	5½	13/16	13	8	27½	6	1
14	21½	16	14	7½	27	6	⅞	15	8½	31	6½	1⅛
15	22¾	17	14½	8	28½	6	⅞	15½	9	33	6½	1 3/16
16	24	18	15	8	30	6½	1	16½	9½	34½	7½	1¼
18	26½	19	16½	8½	32	7	1 1/16	18	10	37½	8	1⅜
20	29	20	18	9½	35	8	1⅛	19½	10½	40½	8½	1½
22	31½	22	20	10	37½	8½	1 3/16	20½	11	43½	9½	1 9/16
24	34	24	22	11	40½	9	1¼	22½	12	47½	10	1⅝
26	36½	26	23	13	44	9	1 5/16	24	13			1 13/16
28	39	28	24	14	46½	9½	1⅜	26	14			1⅞
30	41½	30	25	15	49	10	1 7/16	27½	15			2
32	44	32	26	16			1½	29	16			2⅛
34	46½	34	27	17			1 9/16	30½	17			2¼
36	49	36	28	18			1⅝	32½	18			2⅜
38	51½	38	29	19			1 11/16	34	19			2 7/16
40	54	40	30	20			1¾	35½	20			2 9/16
42	56½	42	31	21			1 13/16	37	21			2 11/16
44	59	44	32	22			1⅞	39	22			2 13/16
46	61½	46	33	23			1 15/16	40½	23			2⅞
48	64	48	34	24			2	42	24			3
50	66½	50	35	25			2 1/16					
52	69	52	37	26			2⅛					
54	71½	54	39	27			2 3/16					
56	74	56	41	28			2¼					
58	76½	58	42	29			2 5/16					
60	79	60	44	30			2 7/16					
62	81½	62	45	31			2½					
64	84	64	47	32			2 9/16					
66	86½	66	48	33			2⅝					
68	89	68	50	34			2 11/16					
70	91½	70	51	35			2¾					
72	94	72	53	36			2 13/16					
74	96½	74	54	37			2⅞					
76	99	76	56	38			2 15/16					
78	101½	78	58	39			3					
80	104	80	59	40			3 1/16					
82	106½	82	60	41			3⅛					
84	109	84	62	42			3 3/16					
86	111½	86	63	43			3¼					
88	114	88	65	44			3 5/16					
90	116½	90	67	45			3⅜					
92	119	92	68	46			3½					
94	121½	94	69	47			3 9/16					
96	124	96	71	48			3⅝					
98	126½	98	73	49			3 11/16					
100	129	100	74	50			3¾					



Reducing Elbow

Long Turn Elbow

Double Branch Elbow

Reducer

* For elbow, 45° elbow, Tee, Cross, Lateral, see Fig. 184, p. 341.

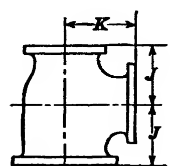
Long-turn elbow and reducer are not specified for heavy pressure larger than 48 in.

straight size. Bull-head tees or tees increasing on outlet, shall have same center-to-face and face-to-face dimensions as a straight fitting of the size of the outlet. For tees, crosses and laterals, 16 in. and smaller, reducing on outlet, use same dimensions as straight sizes of the larger port; 18 in. and larger, reducing on outlet, are made in 2 lengths, depending on the size of the outlet as given in Table 102.

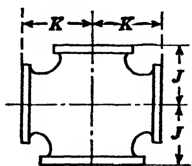
Table 101. Standard Short Body Reducers*

(All dimensions in inches)

Pipe size	Standard weight. Working pressure, 125 lb. per sq. in.							Extra heavy. Working pressure, 250 lb per sq in.				
	Tees and crosses			Laterals				Tees and crosses†		Laterals†		
	Max size of outlets	J	K	Max. size of branches	M	N	O	J	K	M	N	O
18	12	13	15½	9	25	1	27½	14	17	31	3	32½
20	14	14	17	10	27	1	29½	15½	18½	34	3	36
22	15	14	18	10	28½	½	31½	16½	20	37	3	39
24	16	15	19	12	31½	½	34½	17	21½	41	3	43
26	18	16	20	12	35	0	38	19	23			
28	18	16	21	14	27	0	40	19	24			
30	20	18	23	15	39	0	42	20½	25½			
32	20	18	24	...				20½	26½			
34	22	19	25	...				22	28			
36	24	20	26	...				23½	29½			
38	24	20	28	...				23½	30½			
40	26	22	29	...				25	31½			
42	28	23	30	...				26½	33½			
44	28	23	31	...				26½	34½			
46	30	24	33	...				27½	35½			
48	32	26	34	...				29	37½			
50	32	26	35									
52	34	27	36									
54	36	29	37									
56	36	29	39									
58	38	31	40									
60	40	33	41									
62	40	33	42									
64	42	34	44									
66	44	35	45									
68	44	35	46									
70	46	37	47									
72	48	40	48									
74	48	40	49									
76	50	42	50									
78	52	43	52									
80	52	43	53									
82	54	44	54									
84	56	47	56									
86	56	47	57									
88	58	48	58									
90	60	50	61									
92	60	50	62									
94	62	52	63									
96	64	53	64									
98	64	53	65									
100	66	55	67									

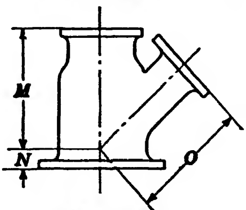


Short Body Reducing Tee



Short Body Reducing Cross

Long Turn Elbow and Reducer are not Specified for Heavy Pressure Larger than 48 In.



Short Body Reducing Lateral

* For minimum metal thickness allowed, see Table 100, page 382.
† Maximum size outlet and branches, same as for Standard Weights.

Table 102. Standard Cast-iron Flange Pipe. Dimensions and Weights.

(R. D. Wood & Co., Philadelphia, Pa.)

Inside diam. of pipe, in.	Outside diam. of flange, in.	Diam. of bolt circle, in.	Number of bolts in flange	Head, 60 ft., pressure, 25 lbs.						Head, 115 ft.; pressure, 50 lbs.						Head, 230 ft.; pressure, 100 lbs.						Head, 350 ft.; pressure, 150 lbs.						Inside diam. of pipe, in.
				Thickness of pipe, in.	Weight, lbs. per foot	Weight, lbs. per length	Weight of single flange, lbs.	Diam. of bolts, in.	Weight of bolts and nuts per joint, lbs.	Thickness of pipe, in.	Weight, lbs. per foot	Weight, lbs. per length	Weight of single flange, lbs.	Diam. of bolts, inches	Weight of bolts and nuts per joint, lbs.	Thickness of pipe, in.	Weight, lbs. per foot	Weight, lbs. per length	Weight of single flange, lbs.	Diam. of bolts, in.	Weight of bolts and nuts per joint, lbs.	Thickness of pipe, in.	Weight, lbs. per foot	Weight, lbs. per length	Weight of single flange, lbs.	Diam. of bolts, in.	Weight of bolts and nuts per joint, lbs.	
3	7½	6	4	0 39	13.0	168	6.1	⅝	1.1	0.41	13.7	177	6 3	⅝	1.1	0.43	14.5	187	6 5	⅝	1.9	0.45	15.2	196	6.7	⅝	1.9	3
4	9	7½	4	0.40	17.3	225	8.6	⅝	1.8	0.43	18.7	242	9.0	⅝	1.8	0 46	20 1	260	9 5	⅝	3.0	0.49	21.6	279	9.9	⅝	3.1	4
5	10	8½	8	0 42	22 3	288	10.2	⅝	3.8	0 46	24 7	318	10.8	⅝	3.8	0 49	26.4	340	11.4	⅝	6.2	0.53	28.7	368	12.0	⅝	6.4	5
6	11	9½	8	0.44	27.8	357	11.8	⅝	3.9	0.48	30.5	391	12.5	⅝	4.0	0.52	33.3	426	13.2	⅝	6.4	0.56	36.0	460	13.9	⅝	6.5	6
7	12½	10¾	8	0.46	33.7	435	15.5	⅝	3.9	0.52	38.3	494	17.0	⅝	4.0	0.56	41.5	534	17.9	⅝	6.5	0.60	44.7	574	18.7	⅝	6.6	7
8	13½	11¾	8	0.47	39.0	503	17.4	¾	3.9	0.53	44.3	559	18.9	¾	4.0	0.59	49.7	637	20.3	¾	6.5	0 64	54.2	693	21.4	¾	6.7	8
9	15	13¼	12	0.49	45.6	592	22.2	¾	6.1	0.55	51.5	666	24.2	¾	6.1	0.62	58.5	754	25.9	¾	9.9	0.68	64.5	829	27.5	¾	10.1	9
10	16	14¼	12	0.51	52.5	679	24.6	¾	9.4	0.58	60.2	776	26 7	¾	9.8	0.65	67 9	873	29.0	⅞	15.8	0 71	74 5	955	30.6	⅞	16.1	10
12	19	17	12	0.54	66.4	869	36.3	¾	9.6	0.63	78.0	1017	40.5	¾	10.0	0.71	88.5	1150	44.1	⅞	16.1	0.79	99 0	1263	47.4	⅞	16.6	12
14	21	18¾	12	0.58	82.9	1081	42.9	⅞	15.4	0.66	94.9	1233	47.0	⅞	15.9	0.77	111.5	1443	52.4	1	24.8	0.86	125.3	1616	56.0	1	25.5	14
15	22¼	20	16	0.60	91.8	1199	48.8	⅞	20.8	0.70	107.7	1401	54.3	⅞	21.5	0.80	123.9	1606	59.4	1	33.1	0.90	140.3	1812	64.1	1	34.4	15
16	23½	21¼	16	0.61	99 3	1302	55.2	⅞	20.8	0.72	118 0	1538	61.2	⅞	22.2	0.84	138.7	1800	67.7	1	33.6	0.95	157.9	2042	73.6	1	34.9	16
18	25	22¾	16	0.65	118.8	1540	57.0	1	31.4	0 77	141.7	1828	63 8	1	32.7	0 90	166 7	2141	70 2	1⅛	45 0	1.02	190.2	2435	75.5	1⅛	47.2	18
20	27½	25	20	0.67	135.8	1769	69.7	1	39.2	0.82	167.4	2168	79.7	1	41.4	0 96	197.3	2544	88 1	1⅛	57.6	1.09	225.0	2890	95.2	1⅛	59.0	20
22	29½	27¼	20	0.72	160.4	2083	79.3	1	40.3	0.87	195.0	2520	89.9	1	42.5	1.02	230.2	2960	99.0	1¼	75.2	1.18	268.1	3431	107 0	1¼	78.8	22
24	32	29½	20	0.75	182.0	2376	95.8	1	40.3	0.92	224.7	2915	109.5	1	42.5	1.08	265.5	3428	121.0	1¼	77.0	1.25	309.4	3976	131.7	1¼	80.6	24
30	38¾	36	28	0.86	260.2	3409	143.1	1⅛	78.4	1.07	325.9	4243	166.0	1⅛	82.3	1.28	392.5	5078	184.2	1⅜	131.6	1.48	456.7	5877	198.7	1⅜	140.0	30
36	45¾	42¾	32	0.96	347.8	4592	209.3	1⅛	91 8	1 21	441.3	5785	244.9	1⅛	98 6	1.46	536 1	6980	273.3	1⅜	156.8	1 71	632.0	8177	296.7	1⅜	166.4	36
40	50¾	47¼	32	1.03	414.2	5522	275.8	1¼	124 5	1 31	330 4	7014	324.6	1¼	134 1	1.59	648 2	8509	365.2	1½	214 4	1.87	767.5	10004	396.9	1½	232.0	40
42	52¾	49½	36	1.07	451.7	6011	295.4	1¼	141.5	1 36	578 0	7628	345 9	1¼	153 7	1.65	705 9	9242	385.4	1½	245.7	1 94	835.5	10866	419 8	1½	265.5	42
48	59½	56	44	1.18	568.8	7602	388.3	1⅜	202.4	1.51	732 8	9710	458.1	1⅜	220.0	1.84	898.9	11811	512.1	1¾	311.3	2.17	1067.1	13912	553.4	1¾	335.5	48

Weights are approximate only; 5 in., 7 in., 9 in., and 15 in. are odd sizes, made to order only; 4 in. to 24 in. usually in stock, regularly made in 12-ft. lengths, faced $\frac{1}{16}$ in. short to allow for gasket. Bolts and gaskets furnished only on order. Flanges are finished approximately 1¼ times thickness of pipe + ¼ in. Drilling is $\frac{1}{16}$ in. larger for ⅝ in. bolts, and ⅛ in larger for ¾ in. bolts and above, than corresponding bolt diam., and all drilling is in fours or multiples thereof. Pressures given are for water; when pipes are required for steam, pressure should not exceed two-thirds that given in table.

Thickness of Cast-iron Water Pipes.* The heads in feet to which the cast-iron pipe thicknesses specified by New England W. W. Assn. conform, are given by the formula:

$$(H + H_r) = \frac{15206.4(t-0.25)}{d}, \text{ or } (P + P_r) = \frac{6600\,(t-0.25)}{d}$$

H = head, ft.; P = pressure, lbs. per sq. in.; t = thickness, in.; d = diam., in.; H_r and P_r being allowances for water hammer. Committee of Am. W. W. Assn. recommends the following values for P_r:

d, inches	H_r, feet	P_r, lbs. per sq. in.
4, 6, 8, 10	277	120
12 and 14	254	110
16 and 18	231	100
20	208	90
24	197	85
30	185	80
36	174	75
42 to 60	162	70

Fanning's formula for thickness in inches, t:

$$t = \frac{(P + 100)d}{1.6S} + 0.333\left(1 - \frac{d}{100}\right)$$

S = ultimate tensile strength of cast iron, lbs. per sq. in.

One of the heaviest cast-iron pipes known is at Scranton, Pa., where there are 3 miles of 48-in. pipe, parts of which are under 600-ft. head; the pipes are 2 in. thick. It has been found by experience that for handling over rough roads, 30-in. cast-iron pipe should not be less than $\frac{7}{8}$ in. thick.†

STRESSES

Stresses Due to Refill. Forces considered: internal static pressure, water hammer, those caused by earth refill over and around pipe.

d = inside diam., in.
t = thickness, in.
F = depth of earth above top of pipe, ft.
s = permissible stress, lbs. per sq. in.; 4000 for cast iron (equivalent to safety factor of 5 for good iron)
W = weight of earth over 1 lin. in. of pipe; assume 115 lbs. per cu. ft., and outside diam. of pipe 1.05 d; $W = 0.84\,Fd$
d_1 = average diameter of shell = about 1.025 d
M = breaking moment normally present from earth
$= \frac{1}{16} W d_1 = 0.0538\,Fd^2$

Resulting maximum circumferential stress in metal is

$S_1 = 6\,M \div t^2 = 0.323\,Fd^2 \div t^2$; stress available for resisting water

* J. T. Fanning, Proc. Am. W. W. Assn., 1903, p. 593.
† M. B. Palmer, Eng. Contr., Nov. 30, 1910.

pressure is 4000 less this amount; of which $\frac{1}{3}$ is allowed for water ram. Hence stress allowable for resisting static pressure is:

$$S_2 = \tfrac{2}{3}(4000 - 0.323Fd^2 \div t^2)$$

H = head in ft. that can be carried by a given stress.

$$H = (12500\,t \div d) - Fd \div t$$

Allowing 0.10 in. in country pipes and 0.25 in city, for inaccuracies of manufacture, etc.

$$t = \frac{d}{25000}\left(H + \sqrt{50000\,F + H^2}\right) + 0.10 \quad \text{(country)}$$

$$t = \frac{d}{25000}\left(H + \sqrt{50000\,F + H^2}\right) + 0.25 \quad \text{(city)}$$

General formula:

$$t = \frac{d}{9.24S}\left(H + \sqrt{27.5\,FS + H^2}\right) + 0.10 \quad \text{(country)}$$

$$t = \frac{d}{9.24S}\left(H + \sqrt{27.5\,FS + H^2}\right) + 0.25 \quad \text{(city)}$$

(A. Hazen, J. N. E. W. W. A., Vol. 25, 1911)

Assuming that pressure due to refill is vertical and uniformly distributed over upper surface of pipe, with no considerable horizontal pressures against pipe, elastic flattening y, in in., or reduction of vertical diam. of relatively thin pipe is:

$y = w(d + t)^4 \div 8Et^3$, whence $w = 8Et^3y \div (d + t)^4$

w = uniformly distributed vertical load per sq. in., lbs.

E = modulus of elasticity of pipe metal = 11,000,000 for cast iron and 30,000,000 for steel and wrought iron.

Data from experience or experiment are few, but assume, where heavy road rollers, etc., do not pass over pipe, that at least $\frac{2}{3}$ weight of refill acts as uniformly distributed vertical load on pipe and produces circumferential bending stresses. Whence maximum stress in outermost fiber, $S = 0.375\,w(d + t)^2 \div t^2$. Assuming refill 6 ft. deep at 115 lbs. per cu. ft., $S = 1.2\left(\frac{d+t}{t}\right)^2$. In general, $(d + t) \div t = 37$ for thinnest large cast-iron pipes and 193 for thinnest large steel pipes. Severe circumferential stresses are caused in cast-iron pipes by settling of several lengths, producing leverage pressures in joints on tops and bottoms of spigots. Assume this load vertical, uniformly distributed over horizontal mean diam. and on effective width of 2 in., and resisted by 12 linear in. of spigot end of pipe; then unit load, $w = 16s\left(\frac{t}{d+t}\right)^2$. Taking 27,000 lbs. per sq. in. as ultimate strength for cast iron in flexure, for 36-in. pipe, 1 in. thick, w = 316 lbs. per sq. in. (E. Kuichling, Jl. N. E. W. W. Assn., Vol. 25, 1911.) See also Bulletin 22, Eng. Expt. Sta. Univ. of Illinois, A. N. Talbot, 1908; E. N., Dec. 15, 1904, W. W. Patch; D. D. Clarke, T. A. S. C. E., 1894, Vol. 38.

Breakages of Cast-iron Pipes, although but a very small percentage of total number of pipes laid, are unfortunately frequent and sometimes disastrous. Common causes are: unequal bearing (a rock or other unyielding object under a portion of the pipe); excessive external load, due to refilling in trench; undiscovered defects of casting; cracks caused in transportation or laying. An unusual cause is assigned by John W. Alvord for breaks in Cincinnati, O., one in a 60-in. main, $1\frac{5}{8}$-in. thick, in good condition, bottom 15.5 ft. below street surface, Dec. 13, 1913, the other in a 48-in. main, Sept. 13, 1914. At and near breaks, each spigot, at some place in its circumference, bore on bottom of adjacent bell, and pipe line was laid on a horizontal curve of large radius. External pressures due to moving ground and other causes were exerted on convex side of curve, throwing pipe into longitudinal compression. In each case a large piece was broken out of one pipe, including the half of the bell on the convex side of the curve, and tapering toward the spigot, being 8 ft. long in the 60-in. pipe and over 10 ft. in the 48-in. (For details see Engg. Contr., Feb. 17, 1915.)

SPECIAL PIPE

Expansion Joint Pipe* is used where exposed to excessive expansion and contraction and for carrying gas or water under high pressure. Molded rubber gaskets are slipped over the spigot ends which are faced and the whole drawn into position by the ring clamp as shown. It is the tightest joint with cast-

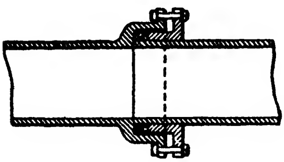


FIG. 209.—Expansion joint pipe.

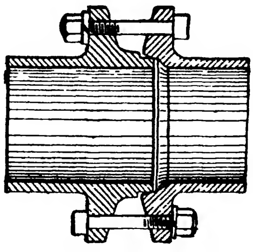


FIG. 210.

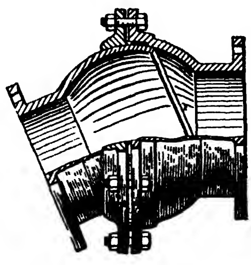


FIG. 211.

iron pipe, for either gas or water, and will hold the highest pressure the pipe will stand, the makers claim.

Table 103. Weight per ft., Expansion Joint Pipe. (12-ft. Lengths)

Size, in	4	6	8	10	12	14	16	18	20	24
Standard: weight per ft., lbs ..	22	34	47	64	82	125	133	160	190	260
Medium: weight per ft., lbs..	24	38	55	73	95	140	150	180	225	300
Extra heavy: weight per ft., lbs.	26	40	60	80	110	145	175	205	250	350

* Jas. B. Clow & Sons, Chicago.

Flexible Ball Joints (Am. Spiral Pipe Wks.) Fig. 211. **Parts are accurately machined, thus making tight joint. Outer bell is reinforced near flange. Retaining flange or ring is constructed to allow a deflection of 20°. It is made to bolt to flange or outer bell, and supports not only body, but entire joint. Flexible joints are discussed on p. 404 also.**

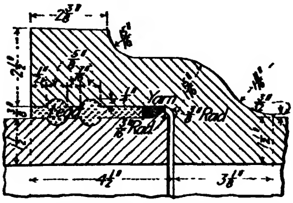


FIG. 212.—Joint for 20-in. high-pressure mains, Manhattan.

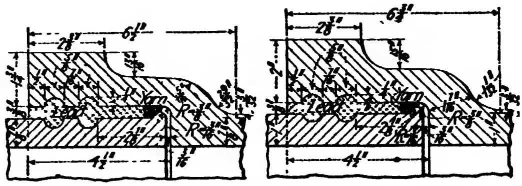
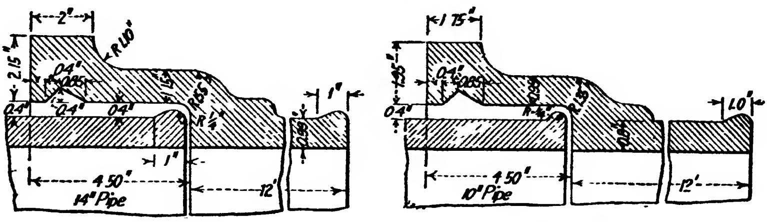


FIG. 213.—Joints for 16-in. high-pressure mains, Coney Island.

Coney Island pipes were tested under 225 lbs ; normal pressure 125 lbs Size 16 in and 12 in Joint dimensions for 16-in pipe, ⅞ and 1 in thick, are shown Extra thickness provides for salt-water corrosion *

High-pressure Fire Pipe. Tests made in New York on an ordinary poured lead joint in a 12-in. cast-iron pipe showed that it held up to 750 lbs. per sq. in., the highest test pressure obtainable.



14-in. pipe 10-in. pipe

FIG. 214.—Joints for high-pressure mains, Oakland, Cal.
(Pressure, 200 lb per square inch E R, July 23, 1910)

Table 104. Thickness of New York City Cast-iron High-pressure Fire Pipe

Diam., in.	Thickness, in.	Unit tensile stress under 300 lbs pressure per sq in	Factor of safety
24	1⅞	1920	10.4
20	1½	2000	10.0
16	1¼	1920	10.4
12	1	1800	11.1
8	⅞	1371	14.6

The 3-way and 4-way special castings are generally of cast steel; a large safety factor being provided.†

Universal Cast-iron Pipe‡ has an iron-to-iron joint consisting of a shallow conical hub with an accurately machined bearing surface and a corresponding short conical machined spigot, the taper of the spigot being ½ deg. sharper than that of the hub. (See Fig. 210.) Each hub and each spigot have a pair of bolt lugs, for ordinary pressures, and two pairs equally spaced for high pressures; in the former case, the bolts are at the ends of the horizontal

* E. R., May 20, 1905.
† E. R., March 25, 1905, p. 344.
‡ Central Foundry Co., 90 West St., N. Y. C.

diameter, as laid, and are useful principally for drawing the pipes together when the line is being constructed. A well laid line will remain tight even if the bolts be removed, except in high-pressure work. Bolts may be preserved by galvanizing, sherardizing or other coating. In making the joints a little white lead in oil is used as a lubricant. This joint is slightly flexible and remains tight under moderate distortion or slight withdrawal of the spigot, consequently the pipe line can endure settlement and expansion without special devices. No packing, gasket, lead nor calking is required, and joints are quickly made. Pipes can readily be put together in shallow water, and no "bell holes" or other enlargements of the trench are needed. Has been used successfully for subaqueous lines, for exposed lines on bridges subject to vibration, and for high-pressure fire service. Pipe lines can be easily taken apart and re-laid. The iron-to-iron joint eliminates or at least greatly reduces electrolysis. Straight lengths may be laid to a curve of 150-ft. radius, and 14-in. pipe has been laid on 88-ft. radius. At Holt, Ala., 72 ft. of 12-in. pipe were left unsupported by a washout and sagged 46 in. but remained quite tight under pumping service until the embankment was rebuilt. In Erie, Pa., similarly a 90-ft. stretch of 12-in. pipe left unsupported by undermining in a flood sagged 4 ft. but remained tight. In Philadelphia high-pressure fire system all joints were tested to 400 lb. while exposed in the trench and required to be absolutely tight; 31 miles were laid 1908 to 1912. Universal pipe was first put on the market in 1900; redesigned in 1908 to increase weight and strength of lugs. A good quality of iron is demanded by the machined joint. Pipes are subjected to hydrostatic test before coating. Pipe is given the usual pipe dip, except the machined surfaces, which are slushed. Specifications may be drawn for pipe lines with no allowance for joint leakage. Even with all bolts removed after laying, pipe lines have remained tight under 150 lb. pressure. Total cost of a pipe line, including trenching, does not differ greatly from that of a corresponding line of the usual bell-and-spigot c.i. pipe with lead joints. The usual varieties of special castings are obtainable. Many manufacturers furnish valves and hydrants with the Universal-joint connection.

Universal Cast-iron Pipe, Dimensions and Weights
(Central Foundry Co., New York City)

Nominal Inside Diameter	Class No. 100, 100 lbs. pressure			Class No 130, 130 lbs. pressure			Class No 175, 175 lbs. pressure			Class No 250, 250 lbs. pressure			Bolt sizes, in.
	Approx thickness, inches	Estimated weight, pounds per Ft.	Estimated weight, pounds per 6-ft. length	Approx thickness, inches	Estimated weight, pounds per Ft.	Estimated weight, pounds per 6-ft length	Approx thickness, inches	Estimated weight, pounds per Ft.	Estimated weight, pounds per 6-ft length	Approx thickness, inches	Estimated weight, pounds per Ft.	Estimated weight, pounds per 6-ft length	
2	. ..	....	.	....	.. .	.	.35	8½	51	.39	9½	57	½ × 3½
3	. . ..	.	. .	. .		.	37	13	78	.42	14½	87	½ × 4
4	.37	18	108	.40	18¾	112½	43	20¼	121½	.45	21¼	127½	⅝ × 5
5	.40	24	144	425	25	150	.45	26	156	.49	29	174	⅝ × 5½
6	.43	30	180	.45	31	186	.47	32	192	.51	35½	213	¾ × 6
8	.47	44¼	265½	.49	46	276	.525	49¼	295½	.58	53¼	319½	⅞ × 6½
10	.50	60½	363	.53	63½	381	.58	67¾	406½	.64	74	444	1 × 7½
12	.53	75½	453	57	80½	483	.62	87	522	.70	97½	585	1 × 8
14	.565	94½	567	60	99½	597	.66	107½	645	.76	124	744	1¼ × 9
16	.60	115½	693	.65	123	738	.72	134	804	.83	156	936	1¼ × 9½
20	.67	166	996	.73	178	1068	.82	196	1176	.94	223	1338	1⅝ × 11¾

Lengths lay full 6 ft. All pipe tested with hydrostatic pressure of 300 lbs per sq in. Gas pipes also tested with compressed air and soap suds. Pipe will be tarred and without tapping bosses unless otherwise ordered. Pipe of any special wall thickness and weight can be promptly furnished.

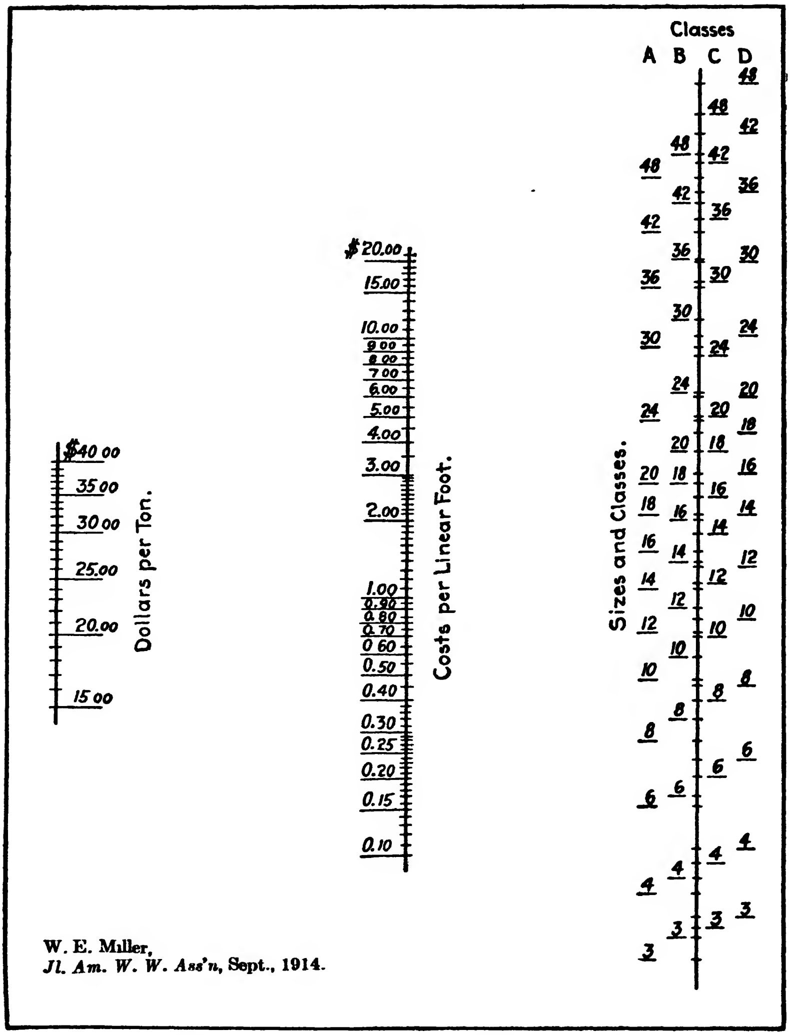


FIG. 215.—Diagram of costs of cast-iron water pipe.
(Classes A, B, C, D, Am W W Ass'n.)

Fig. 215 is useful in making estimates as prices of cast-iron pipe are quoted by makers by the ton, while the engineer is generally interested in costs per linear foot. Most manufacturers use Am. W. W. specifications. Having class and cost per ton, as say, 24″ Class D, and $25.25 per ton, lay a straight-edge through these lines on right and left lines respectively, and read $4 per linear foot on center line.

CHAPTER XIX

DISTRIBUTION SYSTEMS*

General Requirements. Some important requisites of a satisfactory distribution pipe system are well-laid mains of durable material of sufficient capacity to meet all demands with no more than a reasonable drop in pressure; two or more main feeders well located with respect to the system as a whole; all mains inter-connected and so controlled by valves that in case of break or for other reason a small section may be put out of commission quickly, with minimum inconvenience to consumers; master meters registering the quantities delivered into the several districts; protection against electrolysis; ample but not too high pressures; freedom from causes of interruption of flow; a good map showing the position, size, material, depth of cover and date of laying each main, together with the size, location and kind of each valve, hydrant, air valve, blow-off and other appurtenance.

Pipe Size Limits. Except in small villages no permanent street main should be smaller than 6-in; in some communities 8-in. pipe is the established minimum. Possible damage to life and property due to a break increases with size of pipe; usual practice limits street mains to 48-in. For 36-in. and larger mains steel should have serious consideration, especially if pressures are high. It is not prudent to increase even steel pipes in city streets beyond 60 to 66 in. and rarely should cast-iron mains larger than 48-in. be laid for high pressures, although made as large as 72-in. Very large cast-iron pipes, under heavy pressures, are not reliable. Steel pipes have also been known to break or tear unaccountably; but steel is generally reliable until corroded, and almost always gives warning before serious failure. A 60- or 66-in. main is about the largest that can be laid in a city street without undue interference with other structures, especially in an old street. There should be but one large main in any street except for very unusual reasons; otherwise, in case of break, it is impossible quickly to tell which main has failed. Also a break in one would endanger others.

When a cast-iron main fails, a large piece of the pipe blows out, commonly, tending to drain the whole conduit rapidly; steel pipe generally fails by corrosion, causing but small openings, which may leak continuously, unobserved, for a long time, until the loss has become great or the pipe is uncovered; but rarely are there large breaks causing extensive damage. There has been a tendency in recent years to avoid cast-iron pipe over 36 in. diam., or 42 in. at the most. On account of unknown shrinkage and other stresses in large-diam. pipes with thick walls, they are probably less reliable than thinner

* For a valuable general discussion, see "Pipe Distribution Systems" by N. S. Hill, Jr., Jl. Am. W. W. Assn., 1915.

castings. For large conduits some engineers consider it better to use several small cast-iron pipes, or else steel; increasing the number of pipes increases all elements of cost greatly. Board of Water Supply, N. Y., laid cast-iron pipes up to 48 in. in city streets, and 66-in. steel. The latter have lock-bar longitudinal joints for plates $\frac{7}{16}$ in. thick, and riveted joints for thicker plates, determined by capacity of lock-bar machines. (See p. 314.)

Connections at Intersections. Mains which cross should be connected at some of their intersections to allow water more complete circulation throughout the entire system, so that if the supply at any point is temporarily cut off from one direction by closing valves for repairs, or is diminished by excessive demand, it may be maintained from other directions.

Life of Cast-iron Pipe. A section of 12-in. cast-iron service pipe, made by the Washington Iron Works, Newburgh, N. Y., laid in 1854, as a portion of the mains of that city, was dug up in 1909. The casting was originally $\frac{9}{16}$ in. thick, and was buried in ordinary meadow land; long service had produced very little reduction in section; the outside appeared in almost the same condition as when laid, having been protected by a deposit of alkaline scale; the inside showed some pitting and corrosive effects. Cast-iron pipes laid in Paris and Versailles, France, in the 17th century, are reported still in service.

London and Glasgow have records of 120 years' service for C. I. pipe.* Pipes intended for 150 ft. head were used in Lowell, Mass., from 1830 to 1890 under 200 ft. head without signs of failure.† Examination in 1913 of 48-in. pipe laid in 1867 near the large reservoir in Central Park, N. Y. City, in low-lying ground, wet most of the year, showed outside coating practically perfect.‡ In 1897, 1000 ft. of 22-in. pipe, laid in 1817 in Philadelphia, was taken up almost perfect in external conditions. Loss of waterway section due to tubercles was 4.2 per cent.§ See also page 407.

Pressures in Distribution Mains. With properly proportioned distribution systems and comprehensive schemes of fire fighting with fire-engines, hydrant pressures of 30 to 40 lbs. are as good as 60 to 80; additional pressure will not be of material advantage to engines; an engine can pump an adequate supply so long as the pressure on the hydrant is sufficient to deliver water to the engine. It is more necessary, from fire insurance standpoint, to have adequate main capacity than high pressure. Higher pressure allows smaller mains, but in such ratio that it is usually more economical to use larger mains, unless pipes are costly and high gravity pressure easily available. Maintaining high pressures in distribution systems for sprinkler service is questionable; New York Fire Insurance Exchange has refused to consider the city supply as a satisfactory source for sprinkler service, as it is generally metered, and fluctuations are liable, beyond control of the building owner; two independent sources for sprinkler service are demanded before reducing fire rates. In villages and small towns reasonably high pressures in mains will make fire-engines unnecessary.

The pressure on the street mains of a city necessary to give a satisfactory domestic service, depends on hights of buildings. With house pipes of reason-

* M. R. De Pugh, T. A. S. C. E., Vol. 78, 1915.
† J. B. Francis, T. A. S. C. E., Vol. 24, 1891.
‡ W. W. Brush, T. A. S. C. E., Vol. 78, 1915.
§ S. T. Wagner, T. A. S. C. E., Vol. 78, 1915.

able size, static pressure of 20 lbs. on top floor of 6-story building gives good flow; this calls for about 50 lbs. at curb. It does not pay to try to serve sky-scrapers directly from street mains, because of leakages resulting from heavy pressures. Very tall buildings are divided into a convenient number of tiers of several stories each, and each tier provided with pumping apparatus, tanks, etc.

Fire Service Pressures of 100 lbs. and over cause leaks in plumbing, and increase waste; water pumped at high pressure contains myriads of air bubbles as drawn from faucet, making water uninviting for drinking. Few cities in U. S., carrying fire and domestic supply in same pipes, have hydrant pressure of 100 lbs. Average New England pressure is about 75 lbs. at hydrant. This will not supply hose over 300 ft. long. Fire pressures as classified by E. G. Hopson, former chief engineer of Nat. Board Fire Underwriters, Committee of 20: *Low pressure*—below 45 lbs.—sufficient for fire-engine or pump supply but inadequate for other service except to very limited extent or in very low buildings. *Medium pressure*—45 to 70 lbs.—sufficient for moderate streams for inside work for buildings up to 3 or 4 stories with moderate lengths of hose; also for sprinkler supply in buildings of small to medium height. *High pressure*—70 to 100 lbs.—fairly effective stream through hose lengths up to 300 ft., giving excellent auxiliary service to fire department; fairly adequate for sprinklers in all but buildings of excessive hight. (1905.)

Tests of 347 steam fire-engines, 1909–10, by Nat. Bd. of Fire Underwriters, in 45 cities throughout the country, showed the average delivery to be 88 per cent. of rated capacity, based on 100 lbs. net water pressure. At serious fires, long lines of hose, and consequently higher pressures, are required; under such conditions, not more than 50 per cent. of rated capacity can be relied on for any length of time.

PIPE-LAYING: CAST IRON

Bedding Cast-iron Pipes. Cast-iron pipes must be very carefully bedded so as to have uniform support under bottom and must never rest on a bowlder, point of ledge rock, or similar relatively unyielding object. Pipe's capacity to bear refill load can be materially increased by thorough tamping of refill at and below horizontal diam.; but too great dependence on careful refilling is liable to lead to trouble in places where excavations will be made close to the pipe. Probably more breaks of cast-iron pipe lines have been caused by unwittingly or carelessly permitting a pipe to rest on a rock or other hard unyielding object than by any other cause. Neither should large stones be permitted in the refilling close to the pipe. In rock, the trench bottom, after being trimmed, should be covered with a small depth of earth. Each 12-ft. pipe of 16-in. or larger diam. is supported on two short planks, or blocks, one just back of the bell and the other near the spigot end, well bedded on the earth. Each pipe is held in position and brought to bearing on each block by two wooden wedges, of 3×3 or 4×4-in. 12 in. long, according to the size of pipe. Blocking and wedges are placed under special castings and valves also. Sometimes sharp curves are blocked against the side of the trench, or a large mass of concrete is placed to take the reaction.

For large pipes, enlargements of the trench are made at each joint to afford room for calking; these are commonly called bell holes.

Table 104. Trench Details, Solid Lead Joints and Spacing of Air Valves for Cast-iron Pipe Lines

(Metropolitan (Boston) Waterworks Standards)

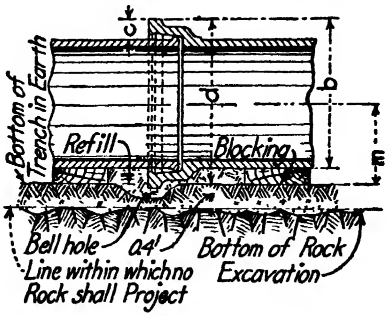


FIG. 216.

Based on New England Water Works Standard Pipe.

Pipe diam., inches	Trench dimensions, ft., Fig. 216: Width of trench at "A line" Earth	Width of trench at "A line" Rock	d	c*	b†	m Earth	m Rock	Wood blocking size, in.	Air valves. Use 1 in. up to following lengths in ft ‡	Solid lead joint: Extra lead lbs	Allowance§
4	2 0	3.5	—	0 14	0 55	—	—	.	Any length	.	
6	2 0	3.5	—	0 15	0 73	—	—			.	
8	2 0	4.0	—	0.16	0 92	—	—	. .		...	
10	2 0	4 0	—	0.16	1 10	—	—	.....		..	
12	2 0	4 0	1 4	0.17	1.28	0 65	0 98	..		9	$0.40
14	2 3	4 0	1 6	0.17	1.47	0.75	1 08	......		11	0.50
16	2 5	4 5	1 8	0 19	1.66	0.90	1 23	2×10×30		20	0.90
20	2 7	4 5	2 2	0 21	2 03	1 10	1.43	2×10×30		23	1 00
24	3 0	5 0	2 5	0 22	2.39	1 25	1.58	2×10×30		28	1 20
30	3 5	5 5	3 2	0 23	2 95	1.60	1 93	3×10×36	5000	45	2.00
36	4 0	6 0	3 7	0 25	3.50	1.85	2 18	3×10×36	3500	54	2 50
42	4 5	6 5	4 3	0 27	4 04	2.13	2.46	3×10×42	2600	78	3.50
48	5 0	7 0	4 8	0 29	4 59	2.40	2 73	3×10×48	2000	88	4.00
54	5 5	7 5	5 4	0 30	5 12	2.70	3 03	3×12×54	1600	115	5.25
60	6 0	8 0	5 9	0 31	5.67	2.95	3.28	3×12×60	1300	130	6 00

* *c* computed for thinnest pipe. † *b* computed for thickest pipe. ‡ Use 1½-in. valve for greater length. § Lead at 4½ cts. per lb.

If joint space is unusually narrow, or pipes are deflected extremely, or great rigidity is desired, or for other special reasons, yarn is sometimes omitted and the joint made solid with lead. In the latter case a little clay luting is used inside the pipe to stop the lead from flowing through, or a single small strand of yarn is driven into the bottom of the joint.

Table 105. Openings of Cast-iron Bell-and-spigot Pipe Joints in Laying Curved Lines with Straight Pipes 12 ft. in Length
(Class E., N. E. W. W. Ass'n Standard)

Joint Opening (ins.) = sine of angle subtended by 12-ft. chord × outside diam. of pipe (ins.)
Deflection of Pipe (ft.) = sine of angle subtended by 12-ft. chord × 12 ft.

Pipe diam.			4″	6″	8″	10″	12″	14″	16″	20″	24″	30″	36″	42″	48″	60″	72″	84″	Sine of angle subtended by 12-ft. chord	
Degree of curve	Radius of curve, ft.	Deflection of pipe in ft	Horizontal opening of joints of pipe in inches																Nat.	Log
	100	1.440	0.59	0.84	1.09	1.35	1.61	1.86	2.11	2.62	3.13								0.120	9.0784
	150	0.960	0.39	0.56	0.73	0.89	1.07	1.24	1.41	1.75	2.09	2.59	3.10						0.0800	8.9024
	200	0.720	0.29	0.42	0.53	0.67	0.80	0.93	1.06	1.30	1.56	1.94	2.32	2.69	3.07				0.0607	8.7829
	250	0.576	0.25	0.35	0.46	0.56	0.67	0.78	0.88	1.09	1.30	1.62	1.94	2.24	2.55				0.0480	8.6811
	300	0.480	0.20	0.28	0.36	0.44	0.53	0.62	0.70	0.87	1.04	1.30	1.55	1.80	2.04				0.0399	8.6010
19	303	0.475	0.19	0.28	0.36	0.44	0.53	0.61	0.70	0.87	1.03	1.28	1.53	1.77	2.02				0.0396	8.5977
18	320	0.451	0.18	0.26	0.34	0.42	0.50	0.58	0.66	0.82	0.97	1.22	1.45	1.68	1.92				0.0375	8.5739
17	338	0.426	0.17	0.25	0.32	0.40	0.48	0.55	0.63	0.77	0.92	1.16	1.37	1.58	1.79				0.0355	8.5501
16	359	0.401	0.16	0.23	0.30	0.37	0.45	0.52	0.59	0.73	0.87	1.08	1.29	1.50	1.70				0.0344	8.5240
15	383	0.376	0.15	0.22	0.28	0.35	0.42	0.48	0.55	0.68	0.82	1.01	1.21	1.40	1.60				0.0313	8.4959
14	410	0.351	0.14	0.21	0.26	0.32	0.39	0.45	0.51	0.63	0.75	0.93	1.12	1.30	1.49	1.79	2.16	2.52	0.0280	8.4464
13	442	0.326	0.13	0.19	0.25	0.30	0.36	0.42	0.47	0.59	0.70	0.87	1.05	1.22	1.39	1.66	2.02	2.34	0.0260	8.4145
12	478	0.301	0.12	0.18	0.23	0.28	0.33	0.39	0.44	0.55	0.65	0.81	0.97	1.13	1.28	1.60	1.94	2.25	0.0251	8.3997
11	522	0.276	0.11	0.16	0.21	0.25	0.31	0.36	0.40	0.50	0.60	0.75	0.89	1.03	1.17	1.47	1.78	2.07	0.0230	8.3615
10	574	0.251	0.10	0.15	0.19	0.23	0.28	0.33	0.36	0.45	0.55	0.68	0.81	0.94	1.06	1.34	1.61	1.88	0.0209	8.3202
9	637	0.226	0.09	0.14	0.17	0.21	0.25	0.30	0.32	0.41	0.50	0.62	0.73	0.86	0.95	1.20	1.45	1.69	0.0188	8.2751
8	717	0.201	0.08	0.12	0.15	0.19	0.22	0.27	0.28	0.37	0.44	0.54	0.65	0.77	0.84	1.07	1.29	1.51	0.0167	8.2236
7	819	0.176	0.07	0.10	0.13	0.16	0.20	0.24	0.25	0.32	0.38	0.47	0.57	0.66	0.74	0.93	1.12	1.32	0.0146	8.1659
6	955	0.151	0.06	0.09	0.11	0.14	0.17	0.20	0.21	0.28	0.32	0.40	0.49	0.56	0.64	0.81	0.97	1.13	0.0126	8.0992
5	1146	0.126	0.05	0.08	0.09	0.12	0.14	0.17	0.19	0.24	0.27	0.34	0.41	0.47	0.54	0.67	0.81	0.95	0.0105	8.0200
4	1433	0.101	0.04	0.06	0.08	0.10	0.11	0.13	0.15	0.18	0.22	0.27	0.32	0.38	0.43	0.54	0.65	0.75	0.00838	7.9231
3	1910	0.075	0.03	0.05	0.06	0.08	0.08	0.10	0.11	0.14	0.17	0.21	0.25	0.29	0.32	0.40	0.49	0.57	0.00628	7.7982
2	2865	0.050	0.02	0.03	0.05	0.05	0.05	0.07	0.08	0.10	0.11	0.15	0.17	0.19	0.21	0.27	0.33	0.38	0.00422	7.6251
1	5730	0.025	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.11	0.13	1.16	0.19	0.00209	7.3211
Depth of bells			3″			3.5″			4″			4.5″		5″		5.5″	5.5″	5.5″		
Outside diam., inches.			4.78	6.92	9.06	11.20	13.3	15.4	17.5	21.6	25.9	32.2	38.5	44.8	51.1	63.8	77.3	90.1		

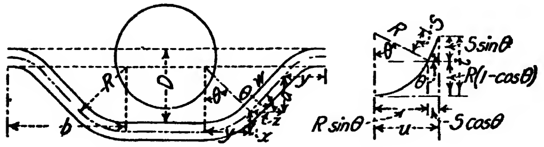
For close computations the first 3 columns relate strictly to the outsides of the pipes on the insides of curves. May be used for both horizontal and vertical curves. Values above heavy broken line may be possible, but are undesirable for good work. (J. H. Lance advises limiting the inside opening of the joint to ½ in. Then D = deflection per pipe length in degrees = maximum degree of curve, divided by 3.33. The maximum curvature used on a 30-in. pipe = 12°30′; and 18° for a 12-in. pipe. By using 6-ft. laying lengths, D = degree of curvature divided by 6.66. E. N., Sept. 29, 1910.)

Table 106. Combinations of Pipe Curves* (Special Castings)
(New England Water Ass'n. Standard)
Angles, Tangents, Chords and Arcs, for Pipes from 24 in. to 60 in. in Diam.

Deflection, angles, ° ′ ″	Diam. in	Combinations of curves†	Radii, ft.	Tangents, ft.‡		Chords, ft.	Length, arcs, ft
5 37 30	30 to 60(a)	1/64	40	1.97	1.97	3 92	3.93
11 15 00	24 to 60	1 1/32	20	1 97	1 97	3 92	3.93
11 15 00	30 to 60	2 1/64	40	3 94	3 94	7.84	7 85
16 52 30	30 to 60	3 1/64	40	5.93	5 93	11 74	11.78
16 52 30	30 to 60	1/64, 1/32	40, 20	4 61	3 30	1.82	7 85
22 30 00	24 to 30	2 1/32	20	3.98	3 98	7 80	7 85
22 30 00	24 to 30	1/16	10	1 99	1 99	3 90	3 93
22 30 00	30 to 60	4 1/64	40	7.96	7.96	15.61	15 71
22 30 00	36 to 60	2 1/32	20	3 98	3.98	7.80	7 85
22 30 00	36 to 60	1/16	15	2 98	2 98	5.85	5 89
28 7 30	30 to 60	5 1/64	40	10 02	10 02	19.44	19 63
28 7 30	36 to 60	1/64, 1/16	40, 15	5 98	4.01	9.71	9 82
33 45 00	24 to 30	3 1/32	20	6 07	6.07	11 61	11 78
33 45 00	24 to 30	1/32, 1/16	20, 10	4 70	3 38	7.74	7 85
33 45 00	30 to 60	6 1/64	40	12 13	12 13	23 22	23 56
33 45 00	36 to 60	3 1/32	20	6 07	6.07	11.61	17 78
33 45 00	36 to 60	1/32, 1/16	20, 15	5 38	4.73	9 67	9 82
39 22 30	30 to 60	7 1/64	40	14 31	14 31	26 95	27 49
39 22 30	36 to 60	1/64, 1/32, 1/16	40, 20, 15	8 40	5 86	13 45	13 74
45 00 00	24 to 30	4 1/32	20	8 28	8 28	15 31	15 71
45 00 00	24 to 30	2 1/16	10	4 14	4 14	7 65	7 85
45 00 00	24 to 30	1/8	5	2 07	2 07	3 83	3 93
45 00 00	30 to 60	8 1/64	40	16 57	16 57	30 61	31 42
45 00 00	36 to 60	4 1/32	20	8 28	8.28	15 31	15 71
45 00 00	36 to 60	2 1/16	15	6 21	6 21	11 48	11 78
45 00 00	36 to 60	1/8	75	3 11	3 11	5 74	5 89
50 37 30	30	1/64, 2 1/16	40, 10	7 55	4 92	11 33	11 78
50 37 30	36 to 60	1/64, 2 1/16	40, 15	9 44	7 25	15 12	15 71
50 37 30	36 to 60	1/64, 1/8	40, 75	6 60	3 75	9 44	9 82
56 15 00	24 to 30	1/32, 2 1/16	20, 10	7 17	5 58	11 26	11 78
56 15 00	24 to 30	1/32, 1/8	20, 5	5 33	2 91	7 36	7 85
56 15 00	36 to 60	1/32, 2 1/16	20, 15	8 94	8.14	15.10	15 71
56 15 00	36 to 60	1/32, 1/8	20, 7 5	6 29	4 30	9 38	9 82
61 52 30	30	1/64, 1/32, 2 1/16	40, 20, 10	10 58	6 59	14 87	15 71
61 52 30	30	1/64, 1/32, 1/8	40, 20, 5	8 91	3 84	11 24	11 78
61 52 30	36 to 60	1/64, 1/32, 2 1/16	40, 20, 15	12 24	9 34	18 57	19 63
61 52 30	36 to 60	1/64, 1/32, 1/8	40, 20, 7 5	9 74	5.21	13 04	13 74
67 30 00	24 to 30	2 1/32, 2 1/16	20, 10	10 19	7 51	14 79	15 11
67 30 00	24 to 30	3 1/16	10	6 68	6 68	11.11	11 78
67 30 00	24 to 30	1/16, 1/8	10, 5	5.10	3 75	7 40	7 85
67 30 00	36 to 60	2 1/32, 2 1/16	20, 15	11.78	10 44	18 49	19 63
67 30 00	36 to 60	3 1/16	15	10 02	10 02	16 67	17 67
67 30 00	36 to 60	1/16, 1/8	15, 75	1 65	5 63	11 09	11 78
73 7 30	30	1/64, 3 1/16	40, 10	10 31	7 57	14 45	15 71
73 7 30	30	1/64' 1/16, 1/8	40, 10, 5	8 78	4 48	10 95	11 78
73 7 30	36 to 60	1/64, 3 1/16	40, 15	13 54	11.25	19 95	21 60
73 7 30	36 to 60	1/64, 1/16, 1/8	40, 15, 7 5	11 24	6.61	14.60	15 71
78 45 00	24 to 30	1/32, 3 1/16	20, 10	10.12	8.40	14.36	15 71
78 45 00	24 to 30	1/32, 1/16, 1/8	20, 10, 5	8 63	5.16	10.88	11 78
78 45 00	36 to 60	1/32, 3 1/16	20, 15	13 27	12.41	19 86	21 60
78 45 00	36 to 60	1/32, 1/16, 1/8	20, 15, 7 5	11 03	7.54	14 52	15 71
84 22 30	30	1/64, 1/32, 3 1/16	40, 20, 10	13 88	9 59	17 63	19 63
84 22 30	30	1/64, 1/32, 1/16, 1/8	40, 20, 10, 5	12 40	6.20	14 40	15 71
84 22 30	36 to 60	1/64, 1/32, 3 1/16	40, 20, 15	16 97	13 91	22 96	25 53
84 22 30	36 to 60	1/64, 1/32, 1/16, 1/8	40, 20, 15, 7.5	14 77	8 22	17 93	19 63
90 00 00	24 to 30	8 1/32	20	20.00	20 00	28.28	31 42
90 00 00	24 to 30	4 1/16	10	10.00	10 00	14.14	15 71
90 00 00	24 to 30	2 1/8	5	5 00	5.00	7.07	7 85
90 00 00	30 to 60	16 1/64	40	40 00	40.00	56 57	62 83
90 00 00	36 to 60	8 1/32	20	20 00	20 00	28 28	31 42
90 00 00	36 to 60	4 1/16	15	15 00	15 00	21 21	23 56
90 00 00	36 to 60	2 1/8	7 5	7 50	7 50	10 61	11 78

* Bends used in Table: 1/64, (angle subtended at center = 5⅝°); 1/32 (11¼°); 1/16 (22½°); 1/8 (45°). Radii, tangents, chords and arcs are of the center line.
† Whole numbers in this column refer to number of curves required. (a) Inclusive in all cases.
‡ Unequal for curves of more than one radius.
The table assumes castings are true to drawings and accurately laid; actually they are likely to vary slightly. Deflections between those tabulated may be made by "opening the joints."

Table 107. Trigonometric Functions of Pipe Curves (Special Castings) for Siphons and Offsets



$a = D - 2x$ $\qquad b = 2y + z$

$w = \dfrac{a}{\sin\theta}$ $\qquad t = R(1 - \cos\theta) + S\sin\theta$

$z = \dfrac{a}{\tan\theta}$ $\qquad u = R\sin\theta + S\cos\theta$

For ¼-curves & 4″ to 12″ ⅛-curves

S In	S Ft.	S sin θ	S cos θ
4	0.33	0.236	0.236
8	0.67	0.667	0.0
10	0.83	0.833	0.0
12	1.00	1.000	0.0

For S see tables giving dimensions of special castings. (N. E. or Am. W. W. Assn. Tables)

Curve	Angle θ	Pipe diam., in	Rad. of curve, ft. R	Natural sine For rad = 1	Natural sine For rad. of curve	Natural cos For rad = 1	Natural cos For rad of curve	$R(1-\cos\theta)$	Natural tan For rad = 1	Natural tan For rad. of curve
¼	90°	4–12	1.33	1.00	1.333	0	0	1.333	∞	∞
		14	1.50	1.00	1.500	0	0	1.500	∞	∞
		16–20	2.00	1.00	2.000	0	0	2.000	∞	∞
		24	2.50	1.00	2.500	0	0	2.500	∞	∞
⅛	45°	4–12	2.00	0.7071	1.414	0.7071	1.414	0.586	1.000	2.000
		14–16	3.00	0.7071	2.121	0.7071	2.121	0.879	1.000	3.000
		20	4.00	0.7071	2.828	0.7071	2.828	1.172	1.000	4.000
		24–30	5.00	0.7071	3.536	0.7071	3.536	1.465	1.000	5.000
		36–60	7.50	0.7071	5.303	0.7071	5.303	2.197	1.000	7.500
1/16	22°–30′	4–12	4.00	0.3827	1.531	0.9239	3.696	0.304	0.4142	1.657
		14–16	6.00	0.3827	2.296	0.9239	5.543	0.457	0.4142	2.485
		20	8.00	0.3827	3.062	0.9239	7.391	0.609	0.4142	3.314
		24–30	10.0	0.3827	3.827	0.9239	9.239	0.761	0.4142	4.142
		36–60	15.0	0.3827	5.741	0.9239	13.859	1.142	0.4142	6.213
1/32	11° 15′	24–60	20.0	0.1951	3.902	0.9808	19.616	0.384	0.1989	3.978
1/64	5° 37′ 30″	30–60	40.0	0.0980	3.920	0.9952	39.808	0.192	0.0985	3.940

This table is useful in field and office for pipe layouts to pass beneath or over sewers, other conduits, streams, etc., or around obstacles.

Laying Out Combined Pipe Bends. When both horizontal and vertical deflections occur at same point in a cast-iron pipe line, but one special need be employed, although it is not uncommon to find each bend made separately, due to difficulty in computing the angle of the skewed special. Two bends give lengths about 1.5 times one bend, thus increasing cost of pipe, as well as friction losses. C. A. Jackson (E. N., Aug. 11, 1910) develops following formulas: "Profile plane" designates vertical plane through center line of either pipe. Fig. 217 shows plan and profile of two pipe lines A and B, to be connected by curve, whose tangent distances are represented in plan and profile by dotted lines.

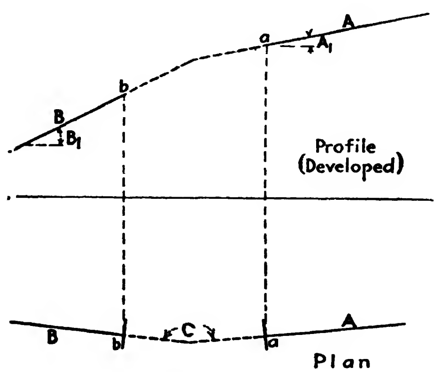


FIG. 217.

Let A_1 = angle of inclination from horizontal of pipe A
B_1 = angle of inclination from horizontal of pipe B
C = horizontal angle between "profile planes"
D = angle of special in plane of bend* (90° for ¼ bend)
L = angle between plane of bend and horizontal plane through horizontal diameter of pipe at b
M = angle between plane of bend and horizontal plane through horizontal diameter of pipe at a
Y = chord of special, = distance from a to b, measured in plane of bend
X = distance horizontally from a to b
Z = difference in elevation between a and b

$$Y^2 = X^2 + Z^2 \tag{1}$$

$$\cos D = \cos A_1 \cos B_1 \cos C + \sin A_1 \sin B_1 \tag{2}$$

$$\cos L = \cos A_1 \sin C \div \sin D \tag{3}$$

$$\cos M = \cos B_1 \sin C \div \sin D = \cos L \frac{\cos B_1}{\cos A_1} \tag{4}$$

Angle D determines what special fits the given case; angles L and M are necessary in drilling bolt holes in flanges. See also p. 335

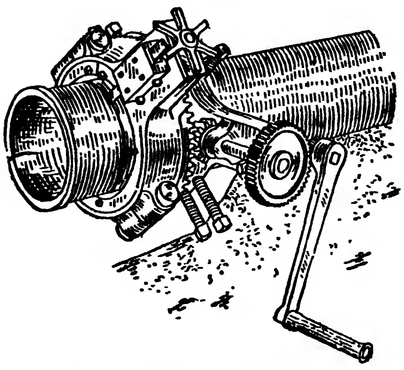


Fig. 218.—French pipe-cutting machine.

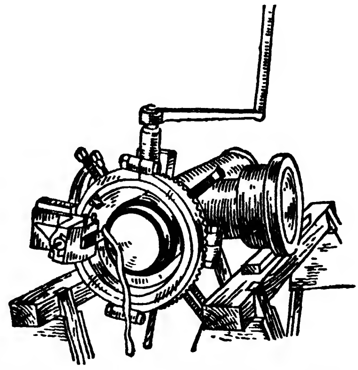


Fig. 219.—French lead joint remover.

Fig. 218.—† No. 1 for use on 2-, 4-, 6-, and 8-in. mains. No. 2 for 10-, 12-, 14- and 16-in. mains. No. 3 for 18-, 20- and 24-in. mains. No. 4 for 30- and 36-in. mains, No. 5, 42- and 48-in. With French pipe cutter pipe can be cut away without breaking or starting next joint; saves material and time. No. 1 is listed at $250 and weighs 164 lbs.; No. 2, $350, 350 lbs.; No. 3, $500, 580 lbs.; No. 4, $700, 750 lbs.; No. 5, $1000, 900 lbs.

Fig. 219.—† No. 1 for use on 4-, 6- and 8-in. mains. No. 2 for 10-, 12-, 14- and 16-in. mains. No. 3 for 18-, 20-, and 24-in. mains. French lead joint remover does not injure pipes, and plugs or valves are saved for future use. Removes lead in one-quarter the time required by old method. Makers combine pipe cutter and lead joint remover at small addition to cost of cutter. Pipes from which beads have been cut can be fixed so that they will not draw out of the bells, by cutting two or three grooves on the end of each pipe so that when leaded into bells the resistance is as great as if beaded. No. 1 is listed at $160 and weighs 210 lbs.; No. 2, $320, 290 lbs.; No. 3, $480, 340 lbs.

* Plane of bend is plane determined by two intersecting center lines of pipe.
† A P. Smith Mfg. Co., Newark, N. J.

Inspection. Pipe laying should be constantly, conscientiously and intelligently inspected, and all pipes, specials, valves and other fittings should be carefully examined for incipient cracks and other defects just before laying.

Table 108. Labor Cost of Cutting Cracked Pipes*

Diam., in.	Men	Time, min.	Cost at 20 cts. per hr.
24	4	25	33 cents
20	4	25	33
16	2	20	14
14	2	20	14
12	2	20	14
10	2	20	14
8	2	10	7
6	2	10	7
4	2	10	7

* Using French pipe cutting machine, A. P. Smith Mfg. Co., Newark, N. J.

Reinforcing Weak Castings. In obtaining an emergency supply for Worcester, Mass., Sept., 1911, no time was available for returning to the pipe foundry 30-in. pipe castings not up to the specified thickness. A local foundry shrunk on circular reinforcing bands of steel, giving a factor of safety of 3 or 4. —(E. R., Oct. 21, 1911, p. 473.)

CALKING PIPE JOINTS

Bell, or Socket, Joints in cast-iron and similar pipes are made with molten lead, lead wool, or shredded lead, or leadite. These fill the outer half of joint depth usually, the remainder having previously been tightly packed with jute yarn or oakum, driven with a yarning tool and hammer. For

Table 109. Weight of Cast Lead and Yarn per Joint in Cast-iron Pipe†

Diam. of pipe, in.	Depth of lead					
	2 in		1½ in		1 in	
	Lead, lb.	Yarn, lb.	Lead, lb.	Yarn, lb	Lead, lb	Yarn, lb
4	6 32	0 251	5 01	0 329	3 16	0 451
6	8 68	0 321	6 88	0 424	4 34	0 579
8	11 03	0 471	8 75	0 605	5 51	0 848
10	13 53	0 578	10 73	0 742	6 76	1 04
12	15 88	0 582	12 60	0.775	7 94	1.05
14	18 67	0 684	14 81	0 912	9 33	1 23
16	21 03	0 642	16 68	0 899	10 51	1 36
18	23 68	0 867	18 78	1 157	11 84	1 56
20	26 33	0 966	20 88	1 288	13 16	1 74
24	31 04	1 137	24 61	1 516	15 52	2 05
30	38 17	1 631	30 37	2 175	19 08	2 94
36	45 28	1.94	36.96	2 58	22 64	3 49
42	68 32	2 92	54 24	3 89	34 16	5 26
48	77 6	3 32	62 20	4 41	38 8	5 97

† Pittsburgh Mfg. Co., Pittsburgh, Pa.

Width of joint = 0.4″ for 3- to 14-in. pipe, ½″ above 14 in.

molten joints a roll of moist clay formed around some yarn has commonly been used to stop the face of the joint during pouring, but the Star pipe jointer (Water Works Equipment Co., N. Y.) and similar devices are preferred by many. Jointers are made for each size of pipe. Jointers are made of fabric packing reinforced by flexible metallic bands, with a gate, or opening for pouring into, and a clamp. Jointers should be kept moist while in use.

Lead Wool Joints. Experiments showed the impracticability of using pneumatic calking with regulation cast lead and yarn. Hand work with lead wool is expensive. Lead wool is calked as follows: Strands ½ in. diam. and 2 to 3 ft. long are rerolled until fairly hard, not as thick as originally, and tapered slightly at each end. Each strand is calked into the joint separately, taking care to break joints. When the face of the hub is reached, the joint is finished as with cast lead. Portable air compressors are used. Under efficient foremen, a calking gang on a 48-in. line, completed 8 joints in 10 hrs.; cost per joint $4.72, including depreciation, against estimated cost $7.11 for hand work, saving 33 per cent. (material not included). Speed also was gained; an important consideration at street intersections where minimum interference with traffic is essential. Two men could do 5 joints per day, as against 2 by hand; 15 per cent. more lead could be put in, giving a tighter

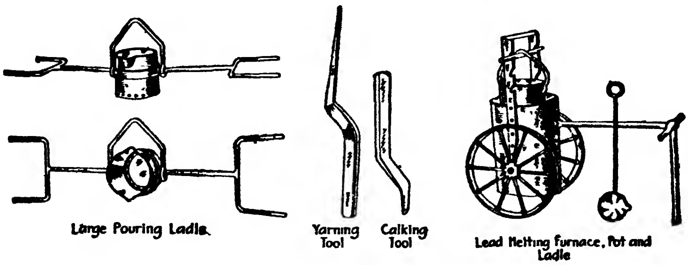


FIG. 220.

joint. Air pressure on receiver gage was about 70 lbs., giving 55 to 65 lbs. at the tool, which was sufficient. The engineer should determine in advance the weight of lead wool to be calked into each joint, according to depth and width of lead space, and then should make sure that no less is put in. This is one of best checks on quality of work done by calkers.—(C. C. Simpson, Pneumatic Calking with Lead Wool of 30-, 36- and 48-in. Mains, Am. Gas Inst., 1910.)

Calking Machine. For calking lead wool or molten lead joints in bell-and-spigot cast-iron water and gas pipes, machines are made by American Calking Machine Co., 94 Warren St., N. Y. C. and by The A. P. Smith Mfg. Co., Newark, N. J., for pipes 12 to 72 in. diam. No bell holes are needed in the trench, and the machine can work partially submerged.

Leadite is a composition of iron, sulphur, slag and salt, finely ground and thoroughly mixed, used for joints on water and gas mains, soil pipe, sewer pipe, etc. Leadite joints, when properly made, will stand 250 lbs. per sq. in., pressure in the pipe. No calking is required. The Water Department, Philadelphia, laid pipe from 48-in. to 4-in., with leadite in 1894; no leaks have been reported. Leadite weighs 118 lbs. per cu. ft., when melted; lead, 708 lbs.—

6 times as much. One ton of leadite will fill more joints than 5 tons of lead. Leadite is put up in 300- to 350-lb. barrels and 100- and 50-lb. bags. It is melted and poured like lead. Only about 400° F. are needed. A gasoline or kerosene furnace is best. Leadite has been used by Pennsylvania Water Co. since 1906 on pipes up to 16 in. diam. and for repairing 42-in. pipes. Ordinary lead has been abandoned excepting for very large pipes, where too rapid pouring and shrinkage of leadite during cooling gave a few leaky joints. Distribution pressures from 75 to 210 lbs. per sq. in. Leadite is easier to manipulate than lead; when cast in a joint, it is hard and vitreous; is more elastic than lead; does not squeeze out as does lead in case of settlement or repeated jars; is especially adapted to work on curves. Leadite joints in 12-in. mains were involved in landslides, which caused but slight leakage; ordinary lead joints would have been blown out. For high pressures it is expedient to use slightly more leadite than called for by Table 111 published by Leadite Co. Labor cost is less for leadite.—(W. C. Hawley, in *Eng. and Contr.*, May 19, 1911.) Following tables show quantity used and costs:

Table 110. Comparative Costs of Lead and Leadite Joints in Cast-iron Pipe

Pipe diam., in.	Lbs per joint		Cost per joint		Saving on material, leadite over lead per joint
	Lead	Leadite	Lead at 5 cts. per lb	Leadite at 10 cts per lb	
4	7 0	2	$0 35	$0 20	$0 15
6	10 0	3	0 50	0 30	0 20
8	13 0	4	0 65	0 40	0 25
10	16 5	5	0 825	0 50	0 325
12	20 5	6	1 025	0 60	0 425
16	29 0	8	1 45	0 80	0 65
20	38 0	10	1 90	1 00	0 90

Table 111. Quantities of Lead and Leadite for Pipe Joints

(Leadite Co., Philadelphia, Pa.)

Size of pipe, in	Lead, lbs	Leadite, lbs	Size of pipe, in	Lead, lbs.	Leadite, lbs
4	6	1½	20	35	8½
5	7½	1¾	24	45	11
6	10	2½	30	60	15
8	12	3	36	80	20
10	15	3¾	40	90	23
12	18	4½	42	100	25
14	22	5½	48	120	30
16	25	6	60	200	50
18	30	7½	72	260	65

Joints assumed, 2½ in. deep

Billé (French Patent) Packing for bell-and-spigot joints (see Fig. 221). Calking is claimed to be effective upon at least 80 per cent. of the depth of joint (in ordinary poured joints, the lead shrinks upon cooling and contact with the iron is assured only within the slight distance affected by calking). The wedging action of wire *C* produces the highest possible contact pressure for the calking strip *A*; the strip *A*, comparatively thin, adapts itself well to irregularities; the wedges *C*, in turn, do likewise, forming a solid calk. The Billé system soft lead joint, 25 mm. (1 in.) wide, recom-

mended for a pressure head of 100 m. (330 ft.), resisted without seepage a pressure of 1900 m. (6270 ft.) under test. The system does away with yarn and permits a corresponding saving in depth of bell. It is claimed to be much cheaper than the common joint.

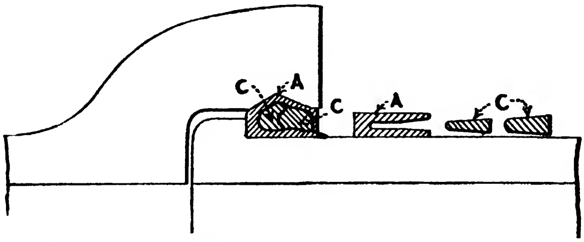
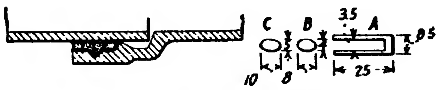


FIG. 221.—Billé packing. *A*, soft lead strip, of U-section, cut and bent to fit spigot end of pipe. *C*, soft lead wedge-shaped wires calked successively into recess of *A*.



Dimensions in millimeters.
FIG. 222.—Billé packing for 330-ft. head.

CAST-IRON WATER PIPES IN FACTORY YARDS*

(Associated Factory Mutual Insurance Cos.)

Suction Pipe. Special care should be taken to lay out each length on a grade rising to the pump so that there will be no summit in which air can collect; use standard weight bell and spigot pipe (Am. Water Works. Assn.); make the joints absolutely tight; test to locate leaks. Where a suction pipe enters a stream or reservoir, lay so as to prevent any possibility of freezing underground or under water.

Depth of Earth Cover over top of the pipe varies from 2.5 ft. in the South, to 5.5 ft. in Canada, New England, Northern New York, and other equally cold sections. In loose, gravelly soil the depth should be greater than in compact clayey soil. Special attention should be given to protection against frost where the pipes pass over raceways or near embankment walls. Such locations should be avoided.

Care in Laying. Pipes should be clean inside when put in trenches; open ends should be plugged when work is stopped to prevent stones rolling inside, or other objects being put in, and to keep out animals. Pipes should bear throughout their length and not be supported by bell ends only. Backfilling should be well tamped under and around pipes to prevent settlement or lateral movement, and should contain no ashes, cinders or other corrosive materials. Rocks should not be rolled into trenches and allowed to drop on pipes. All plugs at blanked openings and all sharp bends in soft ground should either be strapped or secured by masses of concrete.

Locating and Setting Hydrants. Wherever possible, hydrants should be placed about 50 ft. from buildings protected. Where impossible, they may

* Some of the following directions apply also to work in town streets.

be nearer, provided chance of injury by falling walls is small, and men are not likely to be driven off by smoke or heat. Usually in crowded mill yards they can be placed beside low buildings, near brick stair towers, or at angles formed by substantial brick walls, not likely to fall. They should not be placed near retaining walls where there is danger of frost through the wall. In setting the hydrant, about half a barrel of small stones should be placed about the bottom to insure quick drainage from the drip.

Valves. Every pipe entering a building should be provided with an outside valve, so located that it will be accessible for closing in case pipes inside are broken during a fire. Valves in pipes over $2\frac{1}{2}$ in. diam. should, wherever practicable, be provided with indicator posts bearing the name of service controlled. When an indicator post will interfere with teams or cars, use a valve of outside-screw-and-yoke pattern in a brick pit; a manhole should be built so that a wooden cover can be put in about 6 in. beneath the permanent cover, to form an air space to protect against frost. A wrench with long handle should be provided and kept in the pit where it can be reached should the pit be full of water, which is likely at time of a serious fire. The location of the valve should be clearly marked on neighboring buildings, and the cover at all times kept free from dirt and snow.

Check Valves. Every connection to a mill fire system from the public water supply should be provided with a check valve to prevent pumps from forcing into the public mains; the check should be between the valve in the street (usually put in by the waterworks) and the valve in the mill yard, in order that it may be repaired without throwing yard pipes out of service. It should be in a pit, if possible. Every discharge from fire pumps or elevated tanks should have a check valve set underground, or in a brick pit, located so that in case of accident to the pump or an exposed portion of the pipe, the check would prevent water from wasting, besides always keeping water in yard pipes from entering the pump or tank. Where desirable to fill a tank through yard pipes a by-pass may be arranged around the check, to be kept shut excepting when the tank is being filled.

Testing. All mill fire piping when completed should be tested with pumps running 2 hrs., maintaining 150 lbs. pressure; every hydrant should be opened and closed so as to test it, and at same time produce water hammer, as in a fire. System should also be thoroughly flushed out by opening various hydrants wide. Branches to inside sprinkler equipment should also be flushed out before connecting sprinkler risers. System should be tested under water pressure *before joints are covered*, to detect leaks which, even if small, when running continuously, cause large waste of water.

STREAM CROSSINGS

Submerged Pipe Lines.* *Pipe Material.* Submerged pipe lines have been laid in all parts of the world, by ordinary methods, of all ordinary pipe materials except wood staves. The longest line (1915) is the intake of waterworks at Burlington, Vt., 15,480 ft. long, laid on the bottom of Lake Cham-

* See also "Designing of Flexible Joints for Submerged Water Pipe Lines of Cast Iron and Steel," E. Kuichling, Engg. Contr., Apr 15, 1914; also inset, p. 410.

plain. Maximum depth, 120 ft., occurs at New Orleans in Mississippi river. Pipe is usually ordinary cast iron, with the coating common to street mains. Special hard cast iron, riveted steel, riveted wrought iron and ordinary wrought iron have been used, usually coated in the ordinary manner, but in special cases protected by concrete shells, bedded in concrete, or lined with brick. A cast-iron pipe in Boston, in service 50 yrs., was found badly tuberculated inside; in fair condition outside where covered but where uncovered the iron was soft; in places it could be cut to some depth by a knife. One cast-iron pipe carried salt water 20 yrs. without sign of failure.

Joints. All ordinary types of rigid joints have been used, the common type of bell-and-spigot, run with lead, most frequently. Every fifth or sixth joint usually is modified by turning the spigot to a slight taper. The tapered joint is made up with lead on shore, a clamp or strap retaining the lead; the spigot is withdrawn and reentered under water, where a diver calks the lead. Screwed joint of wrought-iron pipe is least satisfactory of rigid joints, due to shearing of threads. Flexible joints are used to fit irregularities of the trench and avoid straining. Majority are ball-and-socket with bearing surfaces of lead on iron. All-iron ball-and-socket joints have been used with ordinary gaskets. These joints allow deflections of 6° to 17°, usually 10°. Flexible jointed cast-iron pipes are usually made in 12-ft. lengths. Possibly there is economy in longer pipes due to weight of bell, but, with small demand for such pipes, is apt to be more than offset by the greater cost for extra long pipes. Shorter lengths are better in some cases to make a more flexible line, or to make less weight to handle. Usually flexible joints are introduced only at intervals of three to six lengths, other joints being rigid, for maximum economy. Steel pipe of large diam. is laid in long lengths, with flexible joints at intervals of 100 to 116 ft. Here connection is not made at a flexible joint, but near it, by a flanged or hub-and-spigot joint. Length of section is usually determined by method of laying.

Foundation. It is usually desirable to cover the pipe after laying; this prevents displacement by currents, flotation, damage from navigation, and increases the durability. Trenches are dredged or washed out. Sometimes it is necessary to put in foundations to prevent settlement where a bottom is soft. Series of pile bents with caps on which the pipe rests and is strapped, wooden blocks laid on bottom of trench, excavating to greater depth and refilling with sand or gravel, strapping timber platforms to pipe in case of steel pipe, and concrete foundations have all been used. The trenches are allowed to silt full, unless danger from anchors calls for prompt refilling. If jeopardized by currents or flotation, refilling is done with care, with various means of anchorage. Pipes have been sunk in mud or sand, after laying, by a water jet or by scouring of the current under the pipe, held just clear of the bottom.

Laying. (*a*) One method of laying pipe is from a construction trestle, built over the trench. The pipe is lowered simultaneously at all points of support, if it has rigid joints. Otherwise, as a joint is completed, it is lowered from the trestle, the other end remaining suspended. This method is not suitable in rough or deep water, or where navigation must not be obstructed.

(*b*) Pipe is sometimes laid on ice and lowered by tackle through an opening in the ice. (*c*) With suitable joints, a pipe can be lowered and connected under water by divers. The pipe is made up on shore in sections, which are floated out over the trench. Flotation is accomplished with casks, through the buoyancy of the pipe when bulkheaded, or it is lowered from a scow. After being connected by the diver, the joint is pulled home by special hydraulic jacks or other means and calked if necessary. (*d*) With flexible jointed pipe in deep water, it is customary to lay and sink the pipe continuously, joints being connected on a scow, and launched from the stern through a chute. The end is fastened securely to the scow. After pouring each joint is deflected sufficiently to break the adhesion between the lead and iron before launching. Care must be taken that the suspended line is not deflected so as to wedge the bell end against the body of the pipe; this can be guarded against by setting the launching ways so that the pipe will "rise from it," unless the safe angle of deflection has been exceeded. In the suspended line, the greatest deflection is at the bottom. This method insures considerable tension in all joints, drawing them tight, but it is difficult in rough weather. Rods running the length of a section are often used to give rigid joints sufficient tensile strength. (*e*) The pipe is also laid in a slide or cradle extending from the scow to the bottom of the trench, built on a curve corresponding closely to shape assumed by pipe line hanging freely and each joint fully deflected. The pipe slides on rails. A plate at the bottom of the slide prevents sinking of slide in trench. Slide may be suspended between scows, or through a well hole in or from one end of a large barge; it is adjustable, so that the curve of the suspended pipe will be tangent to trench bottom. Stern anchors are necessary, due to the forward thrust caused by the weight of the pipe. (*f*) The whole pipe line may be built on shore and hauled into position, buoyancy of pipe, or auxiliary floats being used. When dragged along the bottom, in swift water, a conical pilot is fixed on the first section. Hauling is done by a cable attached to the first section, or run through the pipe and attached to the last length, and to a winch on the farther shore. In still water the pipe can be floated to position and sunk, or pushed from shore. Pipes of all types of joints can be laid this way, except in heavy currents. (See Table 112 for data on these 5 methods.)

Testing. Submarine mains are tested after completion by filling with water under pressure and measuring the leakage by meters. Sections may be tested as completed. With flexible joints, this method tightens the line. Tests with compressed air are also made; the escaping bubbles aid in locating the leaks, if any. Leaks are discovered by divers and calked. Tests show that submerged lines can be made as tight as practical purposes require.

Narrows siphon, Catskill water system, consists of 9800 ft. of 36-in. cast-iron pipe, each laying length of 12 ft. being equipped with a flexible joint of special design. The pipe was laid in a dredged trench, with 8 ft. cover; the greatest depth is 60 ft.; difficulties were increased by the large amount of shipping and swift currents. Pipe was laid continuously from a cradle extending to trench bottom and supported on a scow. Essential features of the joint are: The bell is turned spherically on the interior, and reinforced on the outside by a rolled steel band shrunk on; the spigot has two deep

grooves for retention of the lead, and a turned collar to bear on the bell in order to insure uniform lead space and to carry the weight. Thirty-two gib

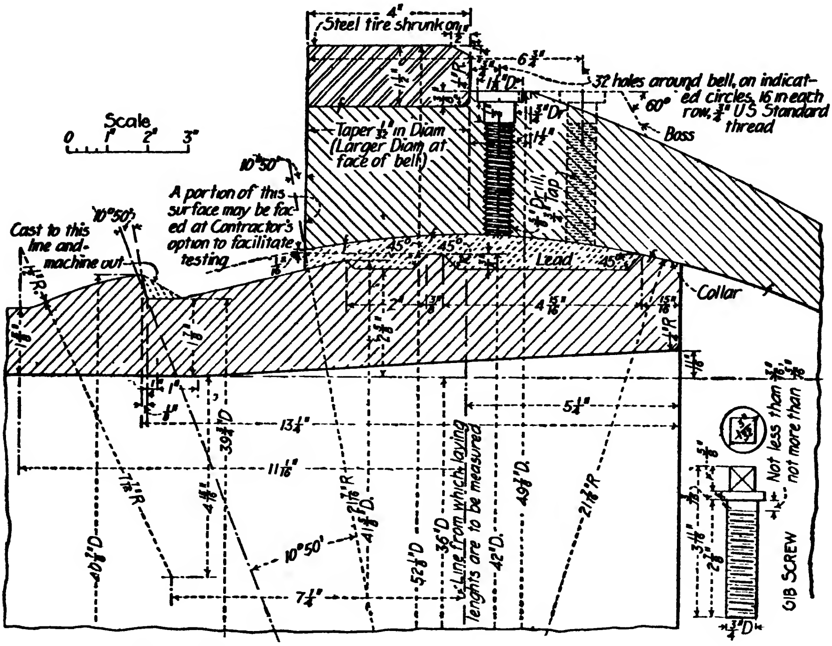
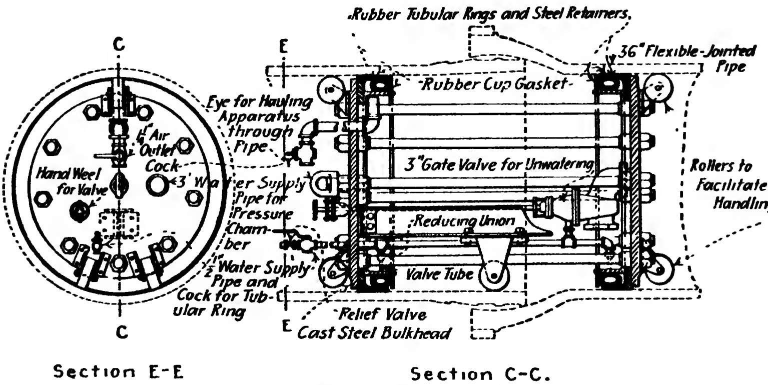


FIG. 223.—Narrows siphon. 36-in. flexible-jointed cast-iron pipe, Catskill Aqueduct.



Rubber tubular rings made and vulcanized in a cast-iron mold as an endless tube.

FIG. 224.—Narrows siphon. Apparatus for testing joints in 36-in. flexible-jointed pipe.

(Pipe rests on slide at about 55° inclination to vertical during tests)

screws are provided in each bell to compress the lead after pouring, by forcing in 144 cold lead slugs 1¾″ × 9/16″ diam. to a total weight per joint of 25.8 lbs. Joints are capable of deflecting 10°. Pipes were tested to 100 lbs., with water

pressure after each joint was made, until it was demonstrated, by the fact that every joint was absolutely tight, that the design and method were dependable. Thereafter a few tests were made of the line as a whole at suitable stages of progress. Pipes are of cast iron, coated inside and out with Bitumastic solution and Bitumastic enamel, applied by American Bitumastic Enamels Co. See Fig. 223 and E. R., Dec. 19, 1914. (See also p. 322.)

Divers. Minimum manhole or other opening in a floor through which a diver can go, on a ladder, is 3 ft. diam. or 3 ft. × 2 ft. 6 in. rectangular; some say 2 ft. 6 in. × 2 ft. 6 in., allowing 6 in. for ladder. Narrowest space in which work can be done, 3 ft. 6 in. to 4 ft.; headroom, 5 ft. (6 ft. is better). Helmet requires 6 in. above man's head. Divers prefer to work with hands lower than head as air escapes at wrists when hands are above head. Divers work in the dark. Can kneel and crawl, if necessary, but some object.

Flexible Lead Pipe. Lead pipe with armoring of locked wire (similar to that used on electric cables), rendering it somewhat flexible and of considerable strength, is manufactured by a German concern (Felten and Guilleaume-Lahmeyer Works, Mülheim-on-Rhine). Lead pipe is surrounded with fiber wrapping, then locked wire armoring, and outside of this asphalted wrapping, if pipe is to be protected from corrosion. Saves joints, can be laid on irregular bottoms without special precautions, under conditions which now call for flexible jointed pipe or trench excavated to uniform grade. Three such pipes, each 1350 ft. long, were laid at Amsterdam, Holland, 1897. Laying took only 35 min. Diam. $2\frac{9}{16}$ in. This was largest diam. made to 1907. In 1907, 6 mi. were being laid at Rio de Janeiro, Brazil, diam. $2\frac{9}{16}$ in., lengths of 650 ft. on a reel. Weight per reel 8 tons. E. N., Aug. 22, 1907.

Pipes in Salt Marshes. Wrought-iron pipe (12 in.) across salt meadows, Atlantic City, down 19 yrs. Action of meadow mud is very severe; after 18 months in it, wrought iron looks as if it had been in acid bath. Deterioration in 19 yrs. from half to full thickness. This was due to presence of spagnum or bog moss, which has acid-producing powers; sometimes classed as peat.* During severe winter, 1904–5, this main froze for first time and 22 lengths broke, probably due to weakness from corrosion.† Thirty-six-in. c.i. pipe laid in 1863 just above surface of salt marshes along Passaic river and supported on piers with no earth covering still showed makers' mark and date in 1914. "W. F. & M. C., 1862." (Warren Foundry & Machine Co.)‡ Wood pipe is best to carry water through salt marshes or salt water. Salt has no decaying effect upon wood, and if the pipe is wound with copper wire, it will last much longer in salt water than cast iron or steel. A 48-in. wood-stave main was laid (1911) across 7 mi. of salt marsh at Atlantic City, N. J., to replace a steel main laid in 1901, badly pitted, and the cast-iron main laid in 1882, also pitted in spots. The wood main has staves 2 in. thick, banded with wrought iron, $\frac{5}{8}$ in. thick, for a pressure of 75 lbs. Long Beach and Elmhurst, L. I., also have wood pipes through salt marshes; the latter is 20 in. diam., 2000 ft. long, operated under pumping pressure.§ See also p. 392 and p. 618.

* W. P. Mason, Proc A W W A, 1902, p. 73.
† Kenneth Allen, Trans. A S. C. E, Vol 78, 1915, p 863
‡ W J. Boucher, Trans A S C. E, Vol. 78, 1915, p 868.
§ T C. Hatton, E N., Mar. 16, 1911.

Concrete can be used successfully to prevent the corrosion of metallic pipes; 1 : 2 : 5 mix, using sand and gravel, densely packed, 4 to 5 in. thick. Joints in the concrete envelope to provide for expansion and contraction should in this latitude be about 30 ft. apart. A thin copper sheet, about 4 in. wide, should be inserted in the concrete at the joints, to prevent leaks. Concrete envelopes have been placed about steel and cast-iron piles, or columns, at Atlantic City and San Francisco, to prevent corrosion from salt water, and have proven satisfactory.

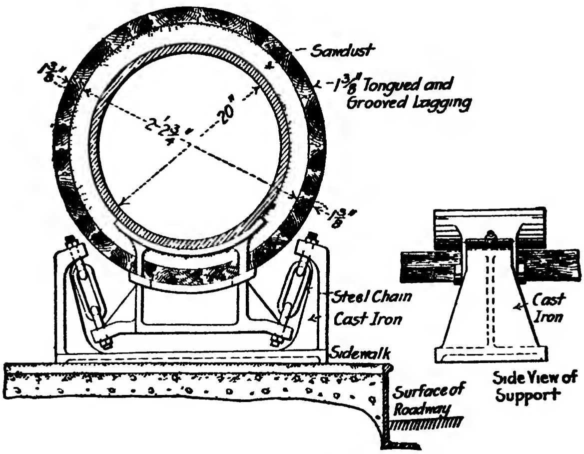


FIG. 225.

Water Main Support on Bridge, Fig. 225, is used to overcome effects due to bridge expansion on pipe if laid directly on bridge and to avoid expansion pipe joints. This saddle support proved satisfactory.—(Louis H. Barker, New York Tunnel Extension of Pennsylvania R. R., Sunnyside Yards, Trans. A. S. C. E., Vol. 69, 1910, p. 143.)

MAINTENANCE AND OPERATION

Wet Connections. Connections to cast-iron mains under pressure are made by special tools without interrupting service. A two-part sleeve is bolted to the main at a point between joints where the connection is desired and joints of sleeve calked with lead or made tight by gaskets. One part of the sleeve has a flanged outlet; to this a permanent flanged valve is bolted. To the other flange of this valve is bolted temporarily a cast-iron dome containing special cutters for cutting from the main a disk of the diam. of the proposed connection and a combination drill and tap, carried by an axial spindle which projects through the stuffing-box in the dome. For connections up to 24 in., hand operation with ratchet levers is used; for larger sizes the machine is driven by a small engine or motor. Valve being open, first drill a hole through the center of the connection and then engage with the tap so as to hold the disk when cut. After the disk has been cut and withdrawn through the valve, valve is closed, dome removed and permanent pipe bolted to valve. Great care is necessary to start the drill straight, other-

wise it may be broken; must not feed too fast or the drill may be broken or pipe cracked, and it may be impossible to remove the disk after cutting. Stuffing-box gland in dome should be loose to permit escape of air until water fills dome. For larger connections, after preparations are complete, about 1 day is needed; smaller connections may be made in half day. Connections from 4 to 36 in. have been made by the A. P. Smith Mfg. Co., Newark, N. J. Waterworks Equipment Co., New York, build an engine-driven pipe-tapping machine for making connections up to 48 in. diam. in the dry.

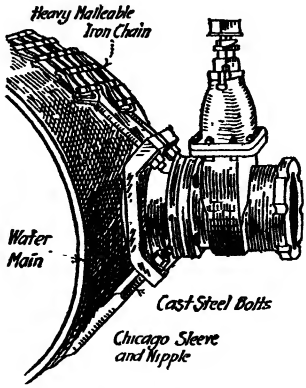


FIG. 226.

Chicago Sleeve and Nipple (Waterworks Specialty Co., Chicago). Shown in Fig. 226. Can be used in connection with or without any standard make of valve and can be attached to mains with pressure on or off with equal ease. Made in all sizes from 3 to 36 in.

Pipe Line Raising. Due to regrading at Seattle, it became necessary to raise a 20-in. cast-iron main under 130 lbs. pressure 17 ft. above its old loca-

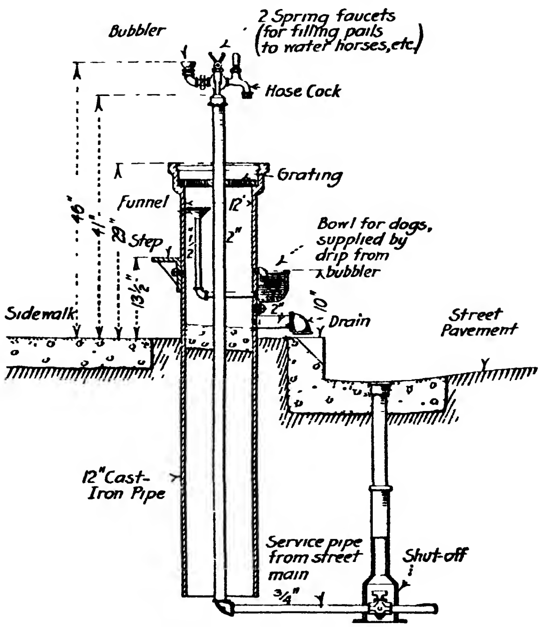


FIG. 227.—Public watering station.

Designed by Frank E. Merrill, Water Commissioner, Somerville, Mass
(Jl N E, W. W. Assn, 1914)

tion, without interruption of flow. Center of 1800-ft. length was raised this amount, the ends remaining in old positions. Jacks and cribbing were employed. A 60-ft. section would be jacked up 1 in., men working to signal to

give even raising. Then the adjacent 60-ft. section would be raised same amount, cribbing placed beneath, etc. Practically no leakage occurred at joints, although some had to be recalked occasionally.—(E. R., July 16, 1910, p. 68.)

Air and Sediment. The capacity of a pipe line may be seriously diminished by the accumulations of air at summits, and of sediment in depressions. Air valves should be placed at summits, and blow-off valves at depressions. See also Air Valves, p. 430.

Operation of Valves. Valves must be closed slowly, and the necessity for this precaution increases with diameters. Otherwise the sudden arresting of the momentum of the water will create great pressure against the pipes in all directions, and throughout the length behind the gate. Unless operated by motor or hydraulic cylinder, valves usually operate much more slowly than is required on this account.

Freezing of Water in Pipes. Unless carrying water at high velocities, cast-iron and steel pipes throughout Northern and Northeastern U. S. and Canada should be laid below the maximum frost line, which extends to depths of 5, 6 or 7 ft. Freezing depends on saturation of material around the pipe, on exposed locality, and on velocity of flow. In freezing, water expands about $\frac{1}{12}$ of bulk, and when rigidly confined exerts an expansive force of about 30,000 lbs. per sq. in. Ordinarily ice causes pressures of about 10,000 lbs. per sq. in. in pipes when not free to expand longitudinally. Pipe should be laid deep (see Table in Jl. N. E. W. W. Assn., Vol. 23, 1909). Good preventive of freezing is "dead air" space around the pipe such as voids in rock fill. Pipes in rock trenches, refilled with dry packing, are less likely to freeze than one in moist, compact soil. Dead ends should be avoided and circulation made as free as possible.

A 6-in. cast-iron pipe laid in 1892, at Springfield, Mass., by regrading of a street was left with but 27-in. cover. Small flow in recent years accelerated freezing. Freezing in 1912 broke a pipe-length; ice was found solid throughout length, except for a cylinder 1 in. in diam. about 1 in. above invert; probably this was last part to freeze. All iron rust and tubercles amassed during 20 yrs. service were concentrated around this 1-in. opening; no rust was left on the periphery.—(E. E. Lochridge, in E. N., Mar. 7, 1912, p. 429.)

Electric Thawing of Submarine Water Main. Large hospital on North Brother Island, New York City, is supplied by a 6-in. cast-iron main, 1700 ft. long, laid on the bottom of the channel. December, 1892, this main froze and had to be relaid after failure of attempts to thaw. Frozen again in Feb., 1912, it was attempted to thaw it by steam. Freezing began where the pipe left the water on the shore of the island, exposed at low tide. After being frozen a month, New York Edison Co. thawed it by electric current, using first 4, and later 6, 100-kw. transformers, 2000 to 200 volts. Pipe was severed on both shores, and current connected 10:15 A. M., March 7, at 800 amp.; water pressure maintained at 80 lbs. in island pipe by electric pumping to help force ice out; in the evening, current was increased to 1000 amp.; on the 8th to 1300 amp.; morning of 9th to 1500 amp. at 400 volts.; 11 A. M. the 10th to 1600 amp. at 368 volts and 2 hrs. later to 1800 amp.; 6:20 A. M.

	Cost per foot									Remarks
	Pipe laying	Laying and testing	Laying, testing, delivered	Dredging	Foundation	Backfill	Total trench	Total except pipe delivered	Total	
38	.....	....	.....	....	0	0	.....	10.47	$21.35	5 anchor cribs at $5000 total.
—	$5.35	....	.....	....	....	....	$6.50	11.85	......	2 pipe lines in one trench.
72	$6.35	....	.....	....	....	....	.....	.....	......	Cost of laying includes valve and crib.
90	2.50	2.50	11.30	....	....	....	.....	.....	......	....
—	———	....	.....	....	0	——	.....	.....	10.46	Trench in rock.
75		3.07	9.82	2.12	0.40	0.91	3.43	6.50	13.25	2 pipe lines in one trench.
—	———	....	....	....	....	....	.....	.....	......	26 bolts tightened in 26 min.
—	———	.. .	....	....		....	.....	....	41.66	Pile foundation.
—	———	...	....	...	....	....	.....	.....	.....	2 Pipe lines in one trench.
—	1.25	....	.....	....	....	....	.....	.....	......	....
—	———	...	.....	....	....	....	.....	.....	4.00	....
06	———	2.50	.....	....	0	?	1.12	3.62	5.58	....
—	0.24	....	.....	....	....	....	.....	.....	......	....
—	———	....	.....	....	....	....	.....	.....	0.75	Trench washed by current.
—	0.10	0.10	.....	0.04	0	0	0.04	0.14	....	....
—	———	....	26.00	....	0	?	18.55	.....	44.55	....
—	———	....	.....	...	0	....	.....	.....	8.00	Cost includes 2 valves.
—	———	...	....	....	0	...	.....	.....	11.00	....
—	———	....	.....	....	0	....	.....	.....	9.30	Cost includes 2 meters.
—	———	....	.....	. ..	0	....	.....	.....	8.70	....
—	———	.. .	.....	....	0	...	.....	.....	9.80	....
—	———		....	...	0	....	.....	.....	8.90	Cost includes 2 meters.
—	———	——	———	——	——	——	Yes	———	2.90	....
—	———	——	———	——	——	——	———	———		....
—	———	——	———	——	——	——	———	———	2 00	Cost includes fish trap.
—	———	——	———	——	——	Yes	———	———	39.94	....
—		——	———	——	——	——	———	———	......	....
11	———	——	———	——	——	——	———	———		....
33	2.71	2.99	8.32	3.24	0	0	3.24	6.24	11.57	....
—	———	——	———	——	——	——	———	———		....
09	———	3.00	6.09		——	——	———	3.00	6.09	....
—	———	——	———	——	0	0	0		5.00	....
—	———	——	———	——	0	0	0	———	3.05	....
—	———	——	———	——	——	——	———	———		Concrete jacket built around pipe before hauling across.

type. *Fa*, Faloon. *Fl*, Flanged. *Fx*, Flexible, miscellaneous. *M*, Metro-
Screwed. *T*, Taper spigot. *W*, Ward type.

m at close of contract. Narrows siphon, see p. 405.

March 12, water began to flow and the pipe was soon thoroughly free. To melt ice in this main 36 times as much heat was required as for same quantity of ice on land; the very cold salt water surrounding the pipe absorbed much heat. —(*Edison Monthly*, April, 1912.)

Cleaning Water Mains.* Small mains are occasionally scraped; a 5 and 6-in. main was scraped at South Molton, England; it was 4.5 miles long, with a theoretic discharge of 0.204 mgd., reduced by 40 yrs. service to 0.115 mgd. Eric pipe scraper was used, consisting of spring-steel blades arranged like a cup, so as to compress to any obstruction on the pipe. These blades enclosed at rear a leather cup, expanding to fill the bore of the pipe, and passing just enough water for flushing. Water pressure on this leather cup drove the machine forward. Attached to the cup by a piston was a plow which scoured. Pipe openings were 573 to 4911 ft. apart. Each machine passed through a section from 3 to 7 times. Pressure gages were used as their variations give an idea of obstructions. Progress of scraper could be detected by sounding the ground with the ear; men lying on ground 3 yds. apart could detect progress of machine in main 8 to 10 ft. deep. Badly made joints often cause stoppage of scraper and occasional breakage of main. Tests of cleaned sections showed 12 per cent. increased capacity.—(E. N., Aug. 31, 1911, p. 259.)

Scraping mains at Dundalk, Ireland, is accomplished by hand and by pressure-driven tools. Hand method is used on pipes 3 to 5 in. diam.; 150 ft. of pipe are laid bare; a 5-ft. pipe length is removed and scrapers fixed to ends of stout cane rods inserted in either direction. During operations, water is allowed to flow freely through the pipe. Service is cut off about 4 hrs. Two types of tools are used: (*a*) a round, chisel-pointed bar with a bow spring on one side, made to fit the circumference of the pipe; (*b*) a spring-steel coil fitted on a ¾-in. round-bar center. Drawing of scrapers by chain and windlass is not recommended. Hand scraping on 3-in. main costs about 1 ct. per lin. ft. Pressure-driven scraper is fitted with adjustable blades and with swivel joint between sets of knives, to facilitate travel around curves; 25 lbs. pressure is necessary to drive 5-in. scraper. Proper rate for driving scraper is about 4 miles per hr. For pipes 12 in. diam., 65 lbs. pressure is required.—(E. R., Sept. 24, 1910, p. 364. Further details in *Eng. and Contr.*, Oct. 12, 1910, p. 315.)

Hartford, Conn.† 33,100 ft. (6¼ mi.) of 20-in. and 30-in. mains were cleaned in 49 days, using 1,655,000 gals. of water. Some incrustations 1 in. thick, nearly around pipe. Some pipe moss (Paludicella) found near reservoir. List prices of National Water Main Cleaning Co. for doing this work in 1912 ranged from 16 cts. per linear foot of pipe for 6-in. pipe, up to 80 cts. for 36-in. Carrying capacity was increased from 50 to 61 per cent.

Distribution Losses. In a large number of cities, almost totally metered, the usual discrepancy is 30 to 50 per cent. between the total pumpage and the meter registration, after allowing for unmetered use and slip of pumps (p. 474). This loss is in the mains and services. A new system, ranging from 4 to 16-in. cast-iron pipe, under 50 lbs. pressure, before the service mains were attached, showed a loss of 1 gal. per min. per mile of main. This test

* For other discussions, see following Volumes of Jl N. E. W. W. Assn · *General*, **5**, 1891, p. 21 and 131, **24**, 1910, p 373; **28**, 1914, p. 70. *St Johns, N. B.*, **13**, 1899, p. 147 and 333. *Boston*, **13**, 1899, p 341

† C. M. Saville, Jl. N. E. W. W. Assn., Vol. 27, 1913.

Table 113. Cleaning Cast-iron Water Mains; Effect on Capacity*

	Location	Diam., in.	Coated?	Length tested, ft	Discharge, gal per min, Before cleaning	Discharge, gal per min, After cleaning	Frictional loss, ft. per 1000 ft., Before cleaning	Frictional loss, ft. per 1000 ft., After cleaning	Values of C in Chézy formula, Before cleaning	Values of C in Chézy formula, After cleaning	Per cent increase, in Discharge	Per cent increase, in Carrying capacity or C	Nature of stoppage
1	Philadelphia	6.0	No	447	364	710	120.0	46.0	34	107	95	215	Mud and corrosion
2	Brooklyn	6.0	No	806	176	623	73.0	—	21	70	254	231	Corrosion
3	Brooklyn	8.0	No	680	349	877	81.0	45.0	18	65	151	234	Corrosion
4	Brooklyn	12.0	No	950	1235	3754	98.0	69.0	22	81	204	265	Corrosion.
5	Brooklyn	30.0	Yes	1363	6050	7350	5.1	Slight	48	...	...	105	Pipe sponge† and corrosion.
6	West Virginia	3.87	Yes	822	—	380	—	137.5	...	96	...	...	Corrosion.
7	West Virginia	5.93	Yes	1050	—	725	—	56.8	...	100	...	...	Corrosion.
8	West Virginia	8.0	Yes	1008	—	1215	—	31.9	...	107	...	...	Corrosion.
9	Perth Amboy, N. J.	4.0	Yes	500	247	477	166.0	145.0	54	110	91	105	Corrosion.
10	Perth Amboy, N. J.	6.0	Yes	601	561	739	65.4	48.4	70	108	32	51	Corrosion.
11	Pittsburgh	8.0	Yes	3264	730	1620	—	—	...	...	121	...	Corrosion
12	Belle Plaine, Ia.	6.0	Yes	5044	248	335	39.8	9.7	40	109	35	174	Lime deposit.
13	Western Pennsylvania	6.0	Yes	822	...	830	—	82.0	...	93	...	...	Corrosion.
14	Western Pennsylvania	4.0	Yes	1100	...	338	—	90.0	...	99	...	...	Corrosion.
15	Troy, N. Y.	6.0	Yes	410	443	703	94.7	46.0	47	105	59	130	Corrosion.
16	Camden, N. J.	4.0	No	420	100	227	—	—	...	...	127	...	Corrosion.
17	Camden, N. J.	8.0	No	—	155	740	—	—		...	375	...	Corrosion.
18	Cumberland, Md.	6.0	Yes	2200	403	802	—	—	...	...	100	...	Slime, mud and corrosion.
19	Madison, Ind.	4.0	No	924	90	262	102.0	90.0	26	75	191	211	Mud and corrosion.
20	Wheeling, W. Va.	6.0	—	—	320	890	—	—	...	...	178	...	Iron corrosion.
21	Lockport, N. Y.	6.0	Yes	1100	315	742	—	—	...	...	135	...	Slime and corrosion.
22	Little Rock, Ark.	6.0	Yes	1090	374	1021	154.6	96.8	31	106	170	344	Corrosion.
23	Pittsburg, Pa.	6.0	Yes	1800	252	671	—	—	...		166	...	Corrosion.
24	Pittsburg, Pa.	4.0	Yes	1834	178	384	—	—	...	...	116	...	Corrosion.
25	Pittsburg, Pa.	4.0	Yes	905	160	485	—	—	...		203	...	Corrosion.
26	New York City	6.0	—	650	435	765	—	—	...	...	76		Corrosion
27	Rochester	10.0	Yes	1045	1125	1740	48.0	30.0	47	90	55	110	Iron corrosion
28	Atlanta, Ga.	30.0	Yes	—	—	—	—	—	—	—	—	—	—
		Pump could drive only 11,880,000 gals per 24 hrs into reservoir before cleaning, and pumping against 110 lbs.; after cleaning line same pump discharged 17,820,000 gals per 24 hrs against same head.											
29	Carbondale, Ill.	8.0	Yes	11574									Clay mud scale.
		Water was pumped through 5000 ft of new 12-in. pipe also and total head pumped against when discharging into open tank 45 ft. higher than pump discharge was 140 lbs. before cleaning; after cleaning total head including frictional loss was 48 lbs.											
30	Centralia, Ill.	12.0	Yes	11000									Mud
		Average rate of consumption, 2 mgd.; average pressure at pumping station was 90 lbs. both before and after cleaning. Average pressure at City Hall before cleaning was 30 lbs., after cleaning 60 lbs.											

*(Hodgman, National Water Main Cleaning Co., Proc. A.W.W.A., 1911.) From data given $C = 80.5\,Q \div D^{\frac{5}{2}} H_f^{\frac{1}{2}}$; Q = discharge, gals. per min.; D = diameter inches H_f = friction loss per 1000 ft.

† A form of spongilla.

indicates that mains may be practically tight.—(E. F. Cole, Technology Quarterly, June, 1907.) (See Table 115, p. 415.)

Pitometer System is a development of the Deacon system, used in England, and for several years in Boston. The pitometer consists of 2 small tubes (bent at the lower ends, and having carefully formed orifices) which are held in a suitable cap, which screws upon a standard 1-in. corporation cock, through which the tubes may be readily introduced into a main. The rod meter is a development of the instrument by which slender tubes are enclosed in an oval sheath, enabling the meter to be introduced into a main through a street connection. Heavy cloth-insertion rubber tubing connects the orifice tubes with a long glass manometer or U-tube, and blow-off cocks are provided to remove air. The U-tube is half filled with a mixture of carbon tetrachloride and gasoline, having a specific gravity of 1.25, and, when in use, water fills the remaining space. The orifices are set to receive the maximum velocity within the main, usually near its center. Velocity is indicated by deflection in the manometer, which, due to the differential action of the water and the slightly heavier insoluble liquid, is 4 times that due to the actual difference of water head on the orifices produced by a flowing stream. The current impinges directly on one orifice, but the other is turned downstream, giving something less than the static head within the main, increasing the difference of pressure produced, which is then multiplied in the U-tube, so that even a low velocity can be read. The pitometer is reliable at velocities as low as 0.5 ft. per sec. Under ordinary pipe velocities, the deflections in the U-tube reach 24 in., requiring no delicate reading. The calibration constant, c, of the orifices has been determined experimentally as 0.80, in the formula, $v = c\sqrt{2gh}$. Thus, $v = 0.8\sqrt{2gD \div 4} = 3.212\sqrt{D}$, in which D is U-tube deflection in ft. After completely traversing a given pipe section on 1 or 2 diam., the ratio of the mean to the maximum velocity is computed for some one rate of flow. The ratio once determined for a given pipe holds good at all velocities. The orifices are permanently set to give center or maximum velocity within a pipe; the mean velocity is read off from a table prepared for a particular size of pipe and velocity ratio. Discharges may be determined by pitometer within 2 or 3 per cent.—(E. F. Cole, Tech. Quarterly, June, 1907.)

Directograph Pitometer (Waterworks Specialty Co., Chicago) consists of a pitot tube and a recording device connected to the tube, which produces graphical records of rates of flow at any instant, in ft. per sec. Accuracy of recorder can be checked at any time by using U-tube. Chart is in form of ribbon or continuous roll showing parallel lines spaced at equal intervals across width of paper denoting ft. velocity per sec., these spaces being subdivided by other parallel lines for fractions. With this form of chart average rate of flow for any given period can readily be determined by planimeter, and where flow is fairly steady can be determined by inspection. When an instrument is to be used at a permanent location, charts can be furnished to read flow in gallons per day. Standard model of recorder has a range between 2 ft. and 6 ft. velocity per sec. and can be used on mains carrying a static pressure up to 125 lbs. Recorders to read higher or lower rates of flow are

built to order. This device has been tested at Worcester Polytechnic Inst. hydraulic laboratory with satisfactory results.

Leakage from Mains.—It is customary to allow 3 gals. per day per ft. (2.67 gals. per lin. ft. of joint per 24 hrs.) for 48-in. cast-iron mains in Brooklyn. A 48-in. steel main in Philadelphia, after being tested and recalked, had leakage of 7000 to 10,000 gals. per mi. per 24 hrs., (based on field joints every 30 ft., leakage per lin. ft. of joint per 24 hrs. = 3.1 to 4.5 gals.) under 160 lbs. pressure. A certain 72-in. steel main gave following test results: Length of section, miles, 3.57, 0.19, 3.43, 0.26, 3.35; leakage per mi., cu. ft. per sec., 0.024, 0.039, 0.034, 0.370, 0.098, respectively. Pressure was 0 to 55 lbs. and 10.8 mi. averaged 0.059 cu. ft. per sec. per mi. (Based on field joints every 30 ft., leakage per lin. ft. of joint per 24 hrs. = 0.000012 mg.) General experience with steel mains is that they tend to grow tighter with age, provided corrosion does not pierce plate. (See also p. 358.)

Tests on 48-in. standard c.i. pipe resulted in a leakage of 7900 gals. per mile per day (1.12 gals. per lin. ft. of joint per 24 hrs.) for a line in Brooklyn under 128 to 163 lbs. test pressure, and leakage of 6700 gals. per mi. per day (1.27 gals. per lin. ft. of joint per 24 hrs.) for a line in Staten Id. under pressures from 47 to 116 lbs.

In removing 50 mi. of cast iron mains in Brooklyn, 90 per cent. being 6 in., and rest 8 to 12 in., only a few dripping joints were found, most of them in a block where the pipe was laid without calking. A 48-in. main sagged 26 in. in 6 lengths without noticeable leakage. On a force main at Ridgewood laid with solid lead joints 3 times the leakage of ordinary joints has occurred.

*Leakage in Cast-iron Water Pipe.**—Geo. T. Deacon, 1894, satisfied himself,

Table 114. Consumption of Water and Percentage Unaccounted for in Well Metered Cities†

City	Per cent of taps metered	Number of years	Consumption per capita, gals	Per cent not accounted for
Brockton, Mass	83 (100)	7 (5)	34	32 (30)
Boston, Mass	------	2	91	34
Cleveland, O. . .	49	1	96	21
Englewood, N. J..	100	1	------	52
Fall River, Mass. .	94 (100)	4 (7)	37	22 (13)
Hackensack, N. J.	100	5	------	40
Hartford, Conn..	99	1	62	39
Lawrence, Mass .	86 (92)	3	46	33 (39)
Milwaukee, Wis	79	1	89	16
Ridgefield, N. J	100	3	163	18
Madison, Wis..	92	8	49	37
Syracuse, N. Y.. .	72	1	108	19
Taunton, Mass..	42	7	57	32
Ware, Mass.	100	1	44	39
Wellesley, Mass.	100	4	52	43
West Orange, N. J.	100	1	------	20
Woonsocket, R. I.	87	1	29	24
Worcester, Mass.	95 (96)	1 (11)	68	42 (28)
Yonkers, N. Y .	97 (100)	6 (1)	83	45 (17)

* E G. Bradbury, Ohio Eng Soc., Jan , 1912

† 1906, J. H. Fuertes. Amount unaccounted for was arrived at by deducting from total quantity the sum of the metered supply and a careful estimate of unmetered consumption, including public uses. German cities show net loss of 6 to 33 per cent. of total consumption. Figures in parentheses were compiled by the Editor for recent years previous to 1915.

Table 115. Total Underground Leakage in Cast-iron Water-pipe Systems in Operation

City	Miles of cast-iron pipe	Size, inches	Leakage, gals per day per mi.	Leakage, gals per day per in. of diam per mi †	Date	Remarks
Hoboken, N. J						
High Service ...	22.37	4–16	1,285 *a*	160*	1883	Metered in and out.
Same	22 37	4–16	12,600 *a*	1,580*	1888	Metered in and out.
Englewood, N. J.	4 78	4–8	1,355 *a*	226*	1888	Metered in and out.
Fall River, Mass.	Whole	System	10,000 *b*	————	1904	96 % metered.
Fall River, Mass	Whole	System	—————	600* *c*	1900	
New York, N. Y	Whole	System	142,000 *d*	————	1900	
New York, N. Y.	Whole	System	—————	13,100* *c*	1900	
Boston, Mass.....	Whole	System	14,187 *d*	————	1900	
Newton, Mass....	Whole	System	3,832 *e*	————	1897	
Brockton, Mass .	Whole	System	6,200 *b*	————	1904	90 % metered.
Ware, Mass .	Whole	System	11,200 *b*	————	1904	100 % metered.
Worcester, Mass.	Whole	System	20,800 *b*	————	1904	94 5% metered.
Wellesley, Mass	Whole	System	3,450 *b*	————	1904	100 % metered
Yonkers, N Y	Whole	System	23,340 *b*	————	1904	100 % metered
Woonsocket, R I	Whole	System	4,370 *b*	————	1904	86 7 % metered.
Milton, Mass	Whole	System	3,110–3,680 *b*	————	1904	Metered in and out.
Belmont, Mass	Whole	System	2,130–4,780 *b*	————	1904	Metered in and out
Melrose, Mass	1 33	4–14	43,770 *b*	6,100	1904	District tests
Chicago, Ill.	25 8	————	202,500 *f*	————	1910	Streets tested before paving
Milwaukee, Wis .	8 72	4–24	75,000 *g*	7,500*	1911	District tests.
Milwaukee, Wis	13 3	6–20	48,800 *g*	4,880*	1911	District tests.
Washington, D C	83 0	3–20	76,500 *h*	10,600	1911	District tests
Providence, R I	1 0	————	39,654 *i*	————	———	Test in mill yard pipe 36 yrs old
Providence, R I	1 0	————	9,263 *i*	————	———	Same after repairing
Providence, R I .	5 57	12–24	2,478 *i*	————	1900	High pressure 114 lbs., fire service 3 yrs old by meter
Brooklyn, N Y	15 39	————	261,500 *j*	————	1910	Summary of district tests for yr
New York, N Y	28 1	12	7,380 *j*	615	1909	By-pass at pumping stations
Akron, O	2 0	6	10,000 *k*	1,667	1911	Street tests
Grandview H'g'ts	5 5	6–12	2.3 *k*	0 31	1911	Metered in and out

* Approximations from data given. *a*, Brush; *b*, Brackett; *c*, Freeman; *d*, Croes; *e*, Kuichling; *f*, Phillips, *g*, Palmer, *h*, Garland and McFarland, *i*, I S Wood, *j*, N Y Rept ; *k*, Bradbury In Washington, D C , for years 1908, 1909, 1910 and 1911, leakage from pipe joints was, respectively, 24, 20, 16 and 37 per cent. of total leakage, average 24 per cent.

† Leakage per in. of diam per mile × y = Leakage per lin ft of joint (time intervals the same), y = 0 0060 for 4" and 0 0075 for 20" pipe (inside diam of bell used)

by investigation of many water supplies in England and other countries, that more than half of all water pumped is, in the average system, lost by leakage. The same year Sir Fredrick Bramwell showed that in over one hundred British cities and towns provided with meters, 66⅔ per cent. of the water was thus lost. W. S. Johnson, 1907, showed an average of 52 per cent. of water accounted for in 3 cities with every tap metered, and an average of 48 per cent. accounted for in 21 cities with an average of 89 per cent. of all taps metered. According to State Board of Health of Massachusetts, 1900, no city of that state having over 90 per cent. of taps metered accounts for over 62 per cent. of water furnished, while one fully metered city finds but 37 per cent. of its supply registered. Broken 4-in. and 6-in. pipes and service pipes running 1000 gals. per hr., have been discovered in Boston, which gave no indications at the surface. Dexter Brackett, 1895, stated that it is not practicable to reduce waste below 15 gals. per capita in large cities, and that this figure can be reached only by universal use of meters, and adoption of thorough methods for detection of underground leaks. Loss from well-constructed systems of 2500 to 3000 gals. per day per mi. was deduced by Emil Kuichling.* C. F. Loweth

*T A. S. C. E , Vol. 38, 1897.

stated, as result of testing several new systems, that leakage can be kept within 60 to 80 gals. per mi. per day, per in. of diam., or 600 to 800 gals. per mi. per day for 10-in. pipe. Length of lead joint per in.-mi. is from 115 to 125 ft., according to number of valves and specials.

Information from 30 Ohio communities showed a domestic consumption per capita of 30 to 225 gals. per day (after deducting railroad and manufactural use). Cincinnati reported estimated leakage 12 per cent.; Springfield 5 per cent.; Toledo 50,000 gals. per day, with total consumption of 16,500,000 gals.; Canal Dover 8000 gals. per day, based on actual test when system was new. In Paris, France (1897), 35 per cent. of the public spring water supply was unaccounted for after making all allowances.

Probably leakage in a new system will not be materially less than 3000 gals. per mi. daily unless carefully tested and all defects remedied. By testing in open trench, under pressure at least 50 per cent. in excess of maximum static pressure expected, recalking all dripping joints and replacing defective pipe, it is possible to reduce leakage to an extremely low amount. With first-class calking, when the men know the work is to be tested in open trench, average loss from 1000 ft. of pipe before testing is about 200 drops per min., equivalent to about 90 gals. per mi. per day. By going over the joints, leakage may be reduced so there is no visible escape of water. Reliance cannot be placed on behavior of a pressure gage in judging tightness of pipe.

For leakage permissible in new work, general practice appears to justify 60 to 250 gals. per day per mi. per in. of diam. Loweth proposes 60 to 80 gals. per in.-mi. Gregory, in improvements to Columbus water supply, specified as in Table 116.

Table 116. Allowable Limits of Leakage from New Cast-iron Water-pipe Lines, Columbus, O.

Pressure 110 lbs per sq in Size, inches	Leakage, allowable gals per hr. per linear ft.	Equivalent in gals. per 24 hrs per mi	Gals per 24 hrs. per in.-mi.
20.	0 08	10,138	507
24. . . .	0 10	12,672	528
36.	0 15	19,008	528

ELECTROLYSIS OF WATER PIPES

Electrolysis, Cause. Electrolysis injures or destroys, sometimes rapidly, cast-iron, wrought-iron, steel and lead mains and service connections. Due to return or stray currents from electric railroads, telephone or other electric service conductors which traverse the pipes as part of the circuit, and at points of departure carry away particles of the metal; also, but less extensively, due to local formation of a battery, as when an electrolyte is present in surrounding soil and steel with mill-scale supplies the other two elements. Earth currents also cause electrolysis, especially in damp ground and along water courses.

Surveys. To determine geographical extent and intensity of conditions causing electrolysis of pipe systems electric surveys are made, measuring

differences of potential at various points between pipes and electric railway or other near-by conductor of current; direction of current with respect to pipes is also determined. Since numerous obscure elements enter most electrolysis problems, surveys are satisfactorily made only by an experienced specialist. Rate and extent of damage vary greatly and are influenced by many local conditions. For example, under a large portion of Newark, N. J., there is light, dry sand very unfavorable to electrolysis; Cincinnati, O., reports no damage, but its electric railways use the double-trolley system; in Altoona, Pa., and many other places trouble is attributed to insufficiently bonded rails of electric roads.

Methods of Prevention. Prevention more or less complete can be secured in several ways: Keep pipes far from electric railways, telephone cables, etc., with electrically resistant soil or other material between; install and maintain complete and adequate metallic circuits for the current, independent of the pipes, such as double-trolley or conduit systems for railways; cover pipes *completely* with insulating coating which cannot be broken down by electric current (coating must remain absolutely perfect over whole surface of pipe, for imperfections are likely to become places of concentrated damage, so that pipe may suffer more than if not so coated). Prof. L. I. Blake, Lawrence, Kan., patented a covering of graphite and paraffin which is a non-ionizable, electrically conducting medium (E. R., 1900, Vol. 42, p. 466). Use insulating joints liberally in pipe lines (see description below). Mitigation of electrolysis can be effected by means suggested above for prevention and by several others: thorough bonding of electric railway tracks; in danger areas near power houses use of overhead return circuits connected to rails at intervals of 500 ± ft.; connection of rails in adjoining negative and positive districts by copper cables; removal of buried return cables in danger districts; metallic connection between pipes and sheaths of telephone cables; special bonding and cross-bonding of electric railway tracks at intersections; periodic thorough inspection of electric railway track returns and other possible sources of trouble; substitution of alternating for direct current; metallic bonding of pipe joints and of pipe line to electric return system, so as to prevent flow of current from pipe to soil. If insulating joints are used in pipe line, negative electric feeders must not be connected. Coatings are generally unsatisfactory; many break down, at least locally, within a few months. Portland cement mortar or concrete coverings are useful if dry, but offer little resistance to current when wet. Following method of insulation is recommended by Professor Ganz, Stevens Institute, as absolutely effective, but too expensive, except for special cases: support pipe on creosoted-wood or glass blocks, in wooden box, so as to leave space of 1 to 2 in. all around; fill space with coal-tar pitch or asphaltum hard enough to remain in place but not crack.

Insulation Joints. Metropolitan W. W., Boston, having serious and persistent electrolysis in extensive pipe systems, installed, during several years, large number of special insulating joints composed of 2 flanged pipes, bolted together with $\frac{1}{2}$-in. gasket of pure rubber between; bolts were covered with $\frac{1}{8}$ in. thick rubber tubing; nuts and bolt heads separated from flanges by rubber washers $\frac{1}{2}$ in. thick. Joints were enclosed in waterproof chambers to prevent

entrance of ground water. Resistance of 48-in. joint when new, pipe filled with water, 100 to 200 ohms. Some of these joints failed in very few years by deterioration of rubber to hard, cinder-like substance, thus destroying insulating value. Beginning in 1909, wooden joints have been placed at intervals of 500 ft. in new pipe lines, at connections with old pipe systems, and to replace rubber insulation joints, sizes 60 in. to 6 in. For these joints pipes are cast without beads on spigot end and without lead grooves in bell; wood ring is placed in bell to separate the pipes, and space ordinarily filled with jute and lead is driven full of pine staves. Wood joint cuts down current flowing on pipe more than 90 per cent. In Providence, R. I., a joint was designed by J. A. McKenna,* which has proved satisfactory since installation in 1909. Fig. 228 shows insulated coupling for 8-in. pipe. Spigot ends of pipe are squared off and outer surface turned smooth. Sleeve is bored out $\frac{1}{16}$ in. and inside edges rounded so as not to shear wood staves when driven in. Staves are little thicker than annular space between pipe and sleeve. Water-tight joint is thus secured, and at same time metallic parts are separated by wood, preventing passage of electric currents. Numerous tests have shown no flow of current through joint.

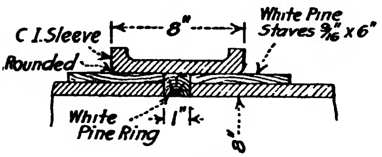


Fig. 228.

Spacing of insulating joints has varied in different places from every joint to one every half mile. In Los Angeles, Cal., filling water-main joints with Portland cement mortar, instead of lead, has given good results; pressures in these mains up to 120 lbs. Ontario Power Co., Niagara Falls, connected 18-ft. steel pipe 6000 ft. long by wire to rails of trolley line and by them to negative poles of dynamo. Somerville, Mass., allowed lighting company to ground secondary wires on water pipes; no ill effects reported. Dresser insulating joint, reported to have given satisfaction in a number of places, is a socket joint packed with cement and sealed with a rubber ring held in place by a bolted flange. (Made by S. R. Dresser Mfg. Co., Bradford, Pa.)

At St. Johns,† N. B., joints in cast-iron mains were made with wood wedges driven tightly in the space usually occupied by the lead and hemp; it was adopted in 1851 as being cheaper and better than lead, and proved so durable that in 1899 the same kind of joint effectually prevented electrolytic action.

Protection of Steel Pipe. At Springfield, Mass., 42-in. lock-bar steel pipe was protected from electrolysis by covering of 2 to 8 plies of "Barrett Specification" felt weighing 14 to 16 lbs. per 100 sq. ft., single thickness, and "Specification" pitch (American coal tar pitch), 30 lbs. each mopping per 100 sq. ft. Felt was cut in strips 12 in. longer than half circumference. Pipe was mopped on top half and felt immediately applied while pitch was hot; strips were laid shingle fashion to break joint every 16 in., for as many plies as required. After covering top half, pipe was turned over and lower half treated in same way. Where pipe was subjected to direct currents, 8 plies were used; as distance from

* E. R., April 8, 1911. † Wyckoff Wood Pipe Co.

current increased number of plies lessened. After pipes were riveted in trench, joints were similarly wrapped.—(W. S. Babcock, in *Engg. Contr.*, Mar. 1, 1911, p. 255.)

Cambridge, Mass. Figure 229 shows effects on 48-in. cast-iron pipe, laid in 1898 near railway power house. Five hundred feet were badly injured by

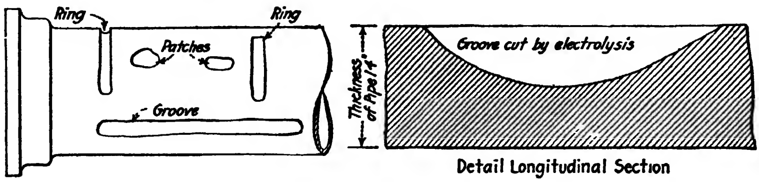


Fig. 229.

electrolysis in patches, rings where rubbed on skids and in grooves. This line was relaid in 1910. A steel rail put in by the railway company to reduce damage to the pipe was badly eaten. Heavy, flexible-joint cast-iron pipes in a river crossing about ½ mi. from power house, were so seriously damaged as to need early replacement (laid 1898). Pipe placed in tunnel and solidly imbedded in concrete was proposed as remedy, but in 1913, advantage was taken of the construction of a masonry bridge at this crossing, Cambridge to Boston, over Charles river to lay a new pipe line over the bridge.

"Perfection" Service Curb Box (S. E. T. Valve & Hydrant Co., New York) will not fill with sand or mud from bottom nor clog with leaves and dirt if cover is left open, or slide away from and damage the valve or cock. Lower section in two parts clamps the service pipe and encases the valve, thus closing lower end of box. It is also provided with a device that supports the valve or cock, maintaining alinement between valve and box, and preventing the valve from turning over or allowing the box to slide away from valve. Top section telescopes the lower, is clamped to it and held in any desired position by two bolts; is provided with a double cover, the outside cover being secured by a special nut placed in a recess in top of cover, which can be operated only by means of a key. Inside cover is provided with a recess similar to the one in outer cover, in which is the actuating nut on the valve rod, operated by same key that removes the cover. If by accident outside cover is broken or lost, box is still closed against entrance of foreign matter. Style 1 is provided with a telescope valve rod which automatically adjusts itself to any position of box; fits any style valve, square, tee or socket; eliminates long solid wrench and does away with removing inside cover. Style 2 is same as Style 1, except the valve rod; a special inside cover is furnished for this box, and is locked in position or removed by a quarter turn of the key; same key for both covers. When inside cover is removed the wrench may be lowered to the valve. If wrench is dropped it will not strike the valve, on account of a division plate placed immediately above it, this plate being provided with an opening of irregular shape corresponding to the opening in the inside cover and to shape of head of wrench; wrench must be slipped through this plate; it cannot be dropped through. Style 2 can at any time be changed to Style 1 by inserting the telescopic rod.—(E. R., Feb. 10, 1912.)

CHAPTER XX

VALVES,* SLUICE GATES AND HYDRANTS

Standard Specifications for Valves. (Very slightly abridged from those adopted by Am. W. W. Assn., at Minneapolis, June, 1913). Requirements for castings, test bars, maker's name and for wrought iron are the same as for hydrants, p. 450.

Composition Metals shall be of the best quality, and, except the stems, shall have a tensile strength of not less than 30,000 lbs. per sq. in., with 15 per cent. elongation in 8 diam., and 15 per cent. reduction of area at the breaking point.

Face Joints shall be faced true and smooth, so as to make, with suitable gaskets, perfectly water-tight joints.

Fitting of all parts shall be such as to make perfect joints and all parts of valves of the same make and size shall be interchangeable. Valves shall open, as specified by the engineer, by turning *left* or *right.*

Bolts and Nuts shall be made from the best quality of refined wrought iron or steel, heads, nuts and threads to be standard sizes.

Kinds and Seats. Valves shall be fully mounted with bronze or other suitable non-corrodible metal. They shall have double disk or made-up gates, with bronze or other suitable non-corrodible metal-mounted wedging devices, or have wedge-shaped gates with double faces and seats, designed to work equally well with pressure on either side of the gate. Gates (or disks) shall be of cast iron with non-corrodible metal faces. These faces shall be machined, dovetailed or driven into corresponding machined grooves in gates (or disks), or riveted on with non-corrodible metal rivets. Seats for composition rings in the body of the valve shall be turned and threaded before the rings are screwed in. Both the seat rings and gate (or disk) rings shall have smooth and true faces, and make perfectly water-tight joints.

Ends and Gears. Valves shall have hub ends suitable for laying with Classes B and C, Am. W. W. Assn., Standard Pipe. All valves 24 in. in diam. and larger shall be geared.

By-passes, where required, shall, unless otherwise specified, be following sizes:

Valve, in.	By-pass, in.
16, 18 and 20	3
24 and 30	4
36 and 42	6
48	8

Weight. Valves without by-passes shall have not less than the following weights:

* Standard valves are made by many firms including: Chapman Valve Mfg. Co , Indian Orchard, Mass.; Coffin Valve Co , Boston, Mass.; Coldwell-Wilcox Co., Newburgh, N. Y.; Crane Co., Chicago, Eddy Valve Co , Waterford, N. Y.; Kennedy Valve Mfg Co., Elmira, N Y.; Ludlow Valve Mfg Co., Troy, N. Y,; Rensselaer Mfg. Co., Troy, N. Y.; R. D. Wood & Co., Philadelphia.

Table 117. Minimum Weight of Valves Without By-passes

Diam., in.	Weight, lb.	Diam., in.	Weight, lb.
3	67	18	1,290
4	85	20	1,650
6	180	24 geared	2,750
8	255	30 geared	5,200
10	400	36 geared	8,500
12	500	42 geared	12,000
14	780	48 geared	18,000
16	900		

Valve Stems shall be of solid brass or other suitable non-corrodible metal, free from defects, and shall have a tensile strength of not less than 45,000 lbs. per sq. in. Threads on stems shall be square, acme, or ½-V and cut in most perfect manner, so as to work true and smooth and in perfect line throughout the lift of the valve. Valve stems at the bottom or base of the thread shall be not less than the following diameters:

Table 118. Minimum Diameters of Valve Stems

(At base of screw thread)

Valve, in.	Diam., in.	Valve, in.	Diam., in.
3	$\frac{44}{64}$	16	$1\frac{28}{64}$
4	$\frac{55}{64}$	18	$1\frac{28}{64}$
5	$\frac{55}{64}$	20	$1\frac{48}{64}$
6	1	22	$1\frac{48}{64}$
7	1	24	$1\frac{62}{64}$
8	1	30	$2\frac{12}{64}$
9	$1\frac{8}{64}$	36	$2\frac{12}{64}$
10	$1\frac{8}{64}$	42	$2\frac{18}{64}$
12	$1\frac{12}{64}$	48	$3\frac{32}{64}$
14	$1\frac{12}{64}$		

Wrench Nut on the stem shall be 2 in. square with an arrow cast on showing the direction which wrench is to turn to open valve.

Painting. All iron work, after being thoroughly cleaned shall be painted throughout with asphaltum varnish, or suitable paint, or dipped in suitable coating material.

Testing. Valves shall be tested for leakage and distortion as follows: On double-disk or made-up gate type the body of the valve shall be drilled and tapped with a hole for pipe and removable plug inserted; through this hole hydrostatic pressure of 300 lbs. per sq. in. shall be applied; the wedge-shaped gate type by an hydrostatic pressure of 300 lbs. per sq. in. applied: first, between one end and the gate; second, between the opposite end and the gate; third, in the bonnet with gate open.

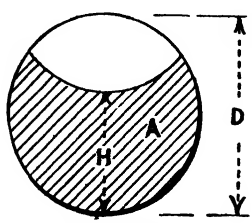


Fig. 230.

Circular Sluice Gates and Gate Valves: Areas of Openings. If the distance a gate or valve has been moved from its seat be known, by indicator or measurement on stem, Table 119 will give the area of opening; or if a gate or valve be opened at uniform speed and total time required to open wide be known, the area of opening at any instant, or any proportion of total time, will be given. Units, ft. and sq. ft. D = diam. and a = area of port of gate or throat of valve. H = distance from bottom of gate or plug to invert of pipe. A = area of opening, Fig 230.

Table 119. Areas of Openings of Circular Gate Valves and Sluice Gates

Time or H/D ratio	$\frac{A}{a}$	Valve Size: 60 in	48 in.	36 in.	24 in.	20 in.	16 in.	12 in.
		Actual areas, A, sq ft.						
0.05	0.063	1.236	0.791	0 445	0.197	0.137	0.088	0.049
0.10	0.127	2 493	1.595	0 897	0.399	0.277	0 177	0.099
0.15	0.190	3.729	2.387	1.343	0.596	0 414	0.265	0.149
0.20	0.253	4.986	3.191	1.795	0.798	0.554	0.354	0.199
0.25	0.315	6.203	3.970	2.233	0.992	0.689	0.441	0.248
0.30	0.376	7.572	4.737	2 664	1.184	0.822	0.526	0.296
0.35	0.436	8.578	5.491	3.088	1.372	0.953	0.610	0.343
0.40	0.495	9.756	6 245	3 513	1 561	1.084	0.693	0.390
0.45	0.553	10.875	6.961	3 915	1.740	1.208	0.773	0.435
0.50	0.610	11.974	7.665	4.311	1.916	1.330	0.851	0.479
0.55	0.663	13.034	8.343	4 693	2.086	1.448	0.927	0.521
0.60	0.715	14.074	9 009	5 068	2 252	1.564	1.001	0.563
0.65	0.765	15.036	9 625	5 414	2.406	1.671	1.069	0.601
0.70	0.811	15 959	10 216	5.746	2 554	1.773	1.135	0.638
0.75	0.855	16 842	10.781	6 064	2 695	1.871	1.197	0.673
0.80	0 895	17.588	11 259	6 333	2 814	1 954	1.251	0.703
0.85	0 932	18.334	11.736	6 602	2.934	2.037	1.304	0.733
0 90	0 962	18.923	12 113	6 814	3 028	2.103	1.345	0.757
0 95	0.987	19 394	12 415	6 983	3.103	2 155	1.379	0.776
1.00	1.000	19.634	12.566	7 068	3 141	2 181	1.396	0.785

In using these tables to compute discharge through gate valves or circular sluice gates it must be borne in mind that almost as soon as the valve or gate is started from its seat there will be a space all around through which water can flow in increasing quantity as the valve or gate moves, before there is any waterway area of the form shown in Fig. 230. Likewise the stem must make a certain number of turns, or it and the valve plug or gate must move a distance somewhat greater than seat facing on valve or gate before H begins to have a value.

Table 120. Areas of Openings of Circular Gate Valves, Sq. Ft.

(Function of turns of valve stem)

Supposing a valve to be raised at a uniform rate by turning the stem at a uniform speed, this table shows the size of the gate opening at any time; also the number of revolutions required to produce the area, based on the number of turns necessary to effect a complete opening

Time or H/D	60-in pipe, No. of turns: 124 (b)	496 (a) (k)	620 (c) (j)	60-in pipe: Area, sq. ft.	48-in pipe, No of turns: 100 (l) (d) (b) (g)	275 (a)	300 (c)	400 (e) (h)	436 (j)	500 (i) (j) (k)	48-in pipe: Area sq. ft
0 05	6	25	31	1 236	5	14	15	20	22	25	0 791
0 10	12	50	62	2 493	10	28	30	40	44	50	1 595
0.15	18	75	93	3 729	15	42	45	60	66	75	2 387
0.20	25	99	124	4 986	20	55	60	80	87	100	3 191
0 25	31	124	155	6 203	25	69	75	100	109	125	3 970
0 30	37	149	186	7 572	30	83	90	120	131	150	4 737
0 35	43	174	217	8 578	35	97	105	140	153	175	5 491
0 40	50	198	248	9 756	40	110	120	160	174	200	6 245
0 45	56	223	279	11 128	45	124	135	180	196	225	6 961
0 50	62	248	310	11 974	50	138	150	200	218	250	7 665
0 55	68	273	341	13 034	55	152	165	220	240	275	8 343
0 60	74	298	372	14 074	60	165	180	240	262	300	9 009
0 65	80	323	403	15 036	65	179	195	260	284	325	9 625
0 70	87	347	434	15 959	70	193	210	280	305	350	10 216
0 75	93	372	465	16 842	75	206	225	300	327	375	10 781
0 80	99	397	496	17 588	80	220	240	320	349	400	11 259
0 85	105	422	527	18 334	85	234	255	340	371	425	11 736
0 90	112	446	558	18.923	90	248	270	360	392	450	12 113
0 95	118	471	589	19 394	95	261	285	380	414	475	12 415
1 00	124	496	620	19 634	100	275	300	400	436	500	12 566

Table 120. Areas of Openings of Circular Gate Valves.—(*Continued*)

Time or H/D	36-in. pipe: Number of turns: 76 (g) (l) (b) (d)	209 (a)	228 (c)	240 (h)	250 (k)(e)	274 (i) (f)	304 (j)	36-in. pipe: Area, sq. ft.	24-in. pipe: Number of turns: 75 (l) (b) (g) (m) (d)	102 (k)	152 (h) (e) (j)	225 (i) (f)	24-in. pipe: Area, sq. ft.
0 05	4	10	11	12	13	14	15	0 445	4	5	8	11	0 197
0 10	8	21	23	24	25	27	30	0.897	8	10	15	22	0 399
0.15	12	31	34	36	38	41	45	1 343	12	15	23	33	0 596
0.20	15	42	46	48	50	55	61	1 795	15	20	30	45	0 798
0.25	19	52	57	60	63	69	76	2.233	19	25	38	56	0 992
0.30	23	63	68	72	75	82	91	2.664	23	31	46	67	1.184
0 35	27	73	80	84	88	96	106	3 088	27	36	53	78	1.372
0 40	30	84	91	96	100	110	122	3 513	30	41	61	90	1.561
0 45	34	94	103	108	113	124	137	3 915	34	46	68	101	1 740
0 50	38	104	114	120	125	137	152	4 311	38	51	76	112	1 916
0 55	42	115	125	132	138	151	167	4 693	41	56	84	124	2.086
0 60	46	125	137	144	150	164	182	5 068	45	61	91	135	2.252
0 65	50	136	148	156	163	178	198	5 414	49	66	99	146	2 406
0 70	53	146	160	168	175	192	213	5 746	53	71	106	157	2.554
0 75	57	157	171	180	188	206	228	6 064	56	76	114	169	2 695
0 80	61	167	182	192	200	219	243	6.333	60	82	122	180	2 814
0 85	65	178	194	204	213	233	258	6 602	64	87	129	191	2.934
0 90	68	188	205	216	225	247	274	6 814	68	92	137	202	3 028
0 95	72	199	217	228	238	261	289	6 983	72	97	144	214	3 103
1 00	76	209	228	240	250	274	304	7 068	75	102	152	225	3 141

Table 120. Areas of Openings of Circular Gate Valves.—(*Concluded*)

Time or H/D	20-in pipe: Number of turns: 64 (g) (l) (m) (b) (d)	128 (e)(h)	192 (f) (i)	20-in pipe: Area, sq. ft.	16-in pipe: Number of turns: 52 (g) (b) (l) (m) (d)	102 (e)	153 (h) (i)	159 (f)	16-in pipe: Area, sq. ft.	12-in pipe: No. of turns: 40 (a) (f) (d) (l) (m)	78 (h)	12-in pipe: Area, sq. ft
0.05	3	6	10	0 137	3	5	8	8	0 088	2	4	0 049
0 10	6	13	19	0 277	5	10	15	16	0 177	4	8	0 099
0 15	9	19	29	0 414	8	15	23	24	0 265	6	12	0 149
0 20	13	26	38	0 554	10	20	31	32	0 354	8	16	0 199
0 25	16	32	48	0 689	13	25	38	40	0 441	10	20	0 248
0 30	19	38	58	0 822	15	31	46	48	0 526	12	23	0.296
0 35	22	45	67	0 953	18	36	54	56	0 610	14	27	0 343
0 40	26	51	77	1 084	21	41	61	64	0 693	16	31	0 390
0 45	29	58	86	1 208	23	46	69	72	0 773	18	35	0 435
0 50	32	64	96	1 330	26	51	77	80	0 851	20	39	0 479
0.55	35	70	106	1 448	29	56	84	88	0 927	22	43	0.521
0 60	38	77	115	1 564	31	61	92	95	1 001	24	47	0 563
0 65	41	83	125	1 671	34	66	100	104	1 069	26	51	0.601
0 70	45	90	134	1 773	36	71	107	111	1 135	28	55	0 638
0 75	48	96	144	1 871	39	76	115	120	1 197	30	59	0 673
0 80	51	102	154	1 954	42	82	122	127	1 251	32	62	0 703
0 85	54	109	163	2 037	44	87	130	135	1 304	34	66	0.733
0 90	58	115	173	2 103	47	92	138	143	1 345	36	70	0 757
0 95	61	122	183	2 155	49	97	146	151	1.379	38	74	0 776
1 00	64	128	192	2 181	52	102	153	159	1 396	40	78	0 785

Ref.	Valve	Pressure	Gearing
a	Double gate	Light	None
b	Double gate	Light	Spur
c	Double gate	Light	Bevel
d	Double gate	Medium	None
e	Double gate	Medium	Spur
f	Double gate	Medium	Bevel
g	Double gate	Heavy	None
h	Double gate	Heavy	Spur
i	Double gate	Heavy	Bevel
j	Double gate, double stem	Heavy	Spur
k	Double gate, double stem	Heavy	Bevel
l	Double gate, outside screw & yoke	Heavy	None
m	Double gate	Extra heavy	None

Valve data are from catalog of Rensselaer Mfg Co., Troy, N Y.

Discharge Through Sluice Gates. Usually the measurement of water by means of sluice gates can be only roughly approximate, unless the gates are designed, set and calibrated with measurement in view. Q (cu. ft. per sec.) = $8 \times c \times$ area in sq. ft. $\times \sqrt{H}$. H is the difference in water levels on the two sides of the gate, in feet, or, if discharge is free, it is the head on the center of the actual opening. The area depends on the position of the gate, whether wholly or partially open. c ranges from 0.60 to 0.80, according as to whether conditions are more nearly like a standard orifice or a short tube. Since the error (*i.e.*, ignorance) is necessarily large, use only two or three significant figures for Q, even for large quantities. (See also Fig. 342, p. 604.)

Stresses in Valve Disks. The following formulas apply to any segment of a sphere, being especially applicable to gate-valve disks of that form: Let r = radius of segment, in.; v = versed sine or middle ordinate, in.; p = hydrostatic pressure, lbs. per sq. in., normal to the surface, acting radially toward the center. Stress at any point in the disk = $p(r^2 + v^2) \div 4v$. Stress in flange = $pr(r^2 - v^2) \div 4v$.—(E. N., Aug. 17, 1911, p. 203.)

Valve Throttling. Ordinary valves should not be used for throttling, especially where tightness is called for when the valve is closed. Throttling a 36-in. valve for several hours each day for 9 yrs. so as to create 15 or 20 ft. higher head (120 to 135± ft.) through another line had the following results: That part of the stem corresponding to ¼ to ⅛ opening of the gate had its threads worn to a feather-edge, so that it slipped through the nut; the seat of the valve was fluted to a maximum depth of $\frac{1}{16}$ in., presenting a rough sandy surface, as if particles of bronze had been removed along the lines of crystallization. The quantity of water delivered by throttling was probably 30 mgd.; velocity for full opening was 6.6 ft. per sec.; approximately 25 ft. for ⅛ opening. The bronze used was probably so-called phosphor bronze, having a tensile strength of 30,000 to 40,000 lbs. per sq. in.

Valve Chattering. On Yakima project, U. S. Reclamation Service, a standard Crane Co. 18-in. gate valve, installed in horizontal position, suffered breakage of tongue due to violent vibratory shocks caused by velocities of over 60 ft. per sec. On a similar valve on same project, care was taken in placing and operating, yet tremendous vibrations occurred when the valve opening was between ⅜ and ¾ diam. An elbow below the valve was pierced with a 1-in. hole which admitted air to the valve chamber, and reduced the vibrations somewhat.—(E. N., Mar. 28, 1912.)

Cylindrical Gate Valves. Dam on Magalloway River, Maine, has 8-ft. undersluices terminating in vertical risers, openings of which are controlled by cylindrical gate valves, such as are used on certain types of turbines. With full opening, under 55-ft. head, the valves each discharge 2000 c.f.p.s. The following advantages are claimed: Quick manipulation; close regulation of discharge; operation by one man; maximum discharge area with minimum head permits drawing reservoir to lowest practical level, at same time maintaining large discharge through gates; rugged construction. Partial tests show coefficient of discharge of 0.89.—(E. R., Mar. 2, 1912, p. 247.)

Narrow Sluice Gates for High Pressure. Large sluice gates under high heads should be made narrow in proportion to the hight so as to secure a reasonable pressure per lin. in. on the seats.

Gate Valves: Power to Open. Information about power required to open large gate valves under high pressures is meager. Following data are reported to have been obtained with care: A pair of 48-in. gate valves with solid wedge-shaped plugs with bronze face rings, Babbitt metal seats in valve body, 3½-in. bronze stems; 30-in. bronze-lined hydraulic cylinders, bolted to 31-in.-high distance or spacing castings, which in turn were bolted to the valve domes; pistons fitted with 2 cup-leather packings. Spindles passed through stuffing-boxes on tops of domes and on bottoms of cylinders. Bronze indicator or tail rods from tops of pistons passed through stuffing-boxes on tops of cylinders (top and bottom refer to valve when in upright position). Valves tested were installed in short 48-in. pipe lines, with axes of valves and cylinders inclined about 20° below a horizontal line. *Valve A*: Pressure per sq. in. on valve when closed; *i.e.*, pressure in 48-in. pipe on upstream side, 41 lbs.; on downstream side about 2 lbs.

Opening of valve, in.	0 (start)	3	6	12	15	18	24	Valve being opened
Pressure in cylinder, lbs. per sq. in.	65	60	51	50	43.5	34 5	24	
Opening, inches	24+ (Started back)	24	18	15	9	3	Final	Valve being closed
Pressure in cylinder, lbs. per sq. in.	12 5	18 5	23	31	40	50	115	

On repeated closing trials, final pressure ranged down to 76 lbs. At 24-in. opening, loud roaring was noticed, but no severe vibration.

Valve B: Upstream pressure, 12.6 lbs.; downstream 2 ±lbs.

	Being opened				Being closed			
Opening of valve, in.	0	3	6	9	9+	6	3	Final
Pressure in cylinders, lbs.	31	17	15	13	11	16	17	20

Pressure in 30-in. Direct-connected Hydraulic Cylinder Required to Open 60-in. Solid-Wedge Gate Valve Under Pressure

Pressure to seat valve, lbs. per sq in.	Pressure on back of valve plug, lbs per sq in.	Starting pressure in cylinder, lbs per sq in	Coefficient of friction
35	52	15	0 22
35	49	15	0 216
50	50	17	0 24

Packing for Stuffing Boxes. Italian hemp well soaked in a mixture of paraffin, vaseline and a little lubricating graphite is durable, and does not corrode the valve parts. Organic fats should be avoided. Several makes of braided-wire and sectional metal packing are on the market; some of them are excellent.

Electrical Operation. Motor operation has proven satisfactory and is much more rapid than by hand. With large gates and valves in some situations the time required for hand work would be greater than emergencies would allow. It has been found necessary in some places to have a man at the switch-

board when a valve is nearly seated, as a precaution against over-seating and buckling of stem. This failure of motors to cut-off when valves are seated is caused by the contacts connected with the automatic shut-offs or limit switches; if these contacts are set so that an opening valve cuts off correctly, it seems impracticable to set them so that a closing valve will also shut off correctly. It has been a common error to provide motors of insufficient power to *start* the

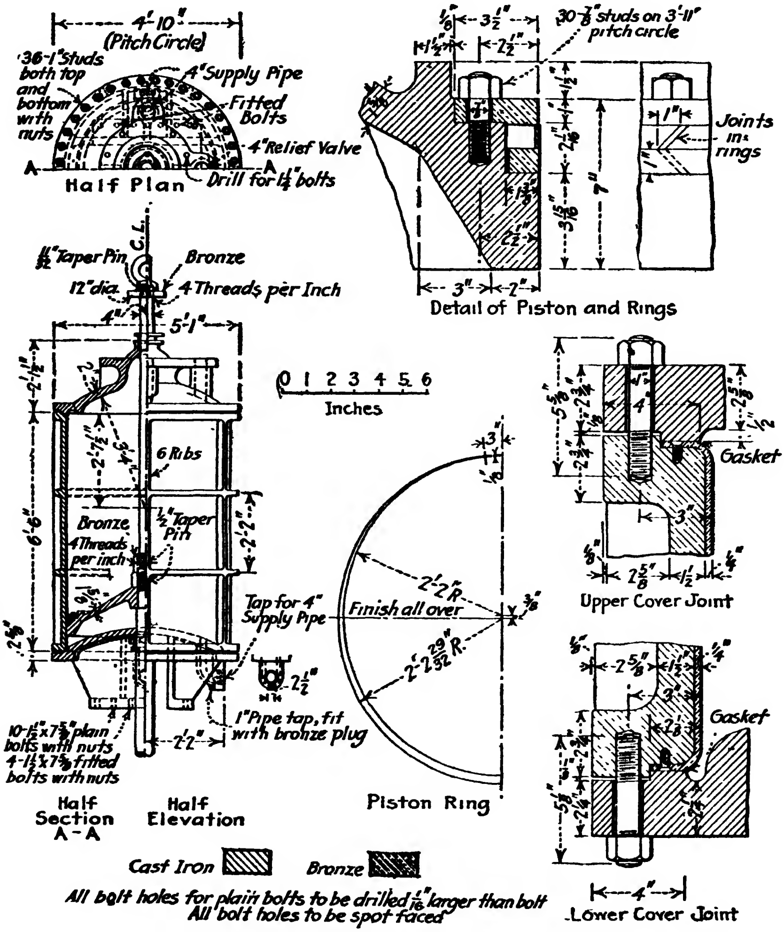


FIG. 231.—Hydraulic cylinder for large gate valves.
(Catskill Aqueduct)

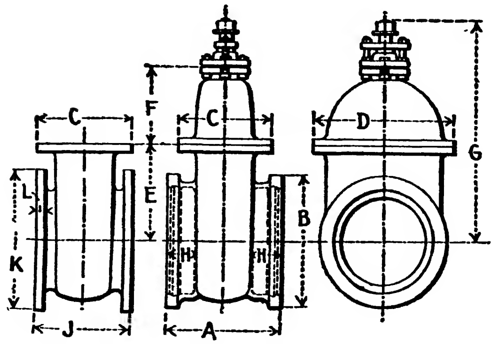
valves after having been closed a long time. Valves require more power to move them, generally, as they grow older.

Hydraulic-lift Valves: *Liquids in Cylinder.* Investigation has shown some form of mineral oil having a low cold test preferable to any mixture of glycerine or alcohol and water, or any brines used in refrigerating work, as former do not remain stable in composition, and latter rapidly corrode metal. At Spot Pond gate-house, Boston, ordinary ice-machine oil (zero cold test paraffin oil, Standard Oil Co.) is used. This begins to solidify at 2° or 3° below zero F.; it is not

suitable for exposed positions, but has answered perfectly in gate-houses, where temperature is kept above zero. For very severe exposures, such as hydraulic cylinders on the hoisting mechanism of Charlestown bridge, Boston, a preparation of Russian petroleum, "Hydrol" (Davies, Rose & Co., Boston), has been satisfactory since 1900, being subjected to temperatures of 10° or more below zero F. According to tests of Boston Bridge Dept., at 24° below zero, this oil has consistence of lard. In addition to anti-freezing qualities, it is a good lubricant, and will not corrode metals. Hydrol is pure hydrocarbon, colorless, tasteless, odorless, non-oxidizable, neutral. Sp. gr. at 60° = 0.865. Does not solidify at 0° F.; at −5° F. it is viscid, slow-flowing. Frankfort arsenal tests of viscosity: 0° F., 125.6 sec.; 75° F., 68.6 sec.; 100° F., 42.4 sec. It is difficult to get oil-tight joints on hydraulic cylinders. Rubber packing and all ordinary pipe-joint compounds are attacked by oil. Oil or shellac serves best for getting tight screwed joints; paper or leather saturated with a mixture of glue and glycerine, for packing flanged work.

Requirements for hydraulic cylinder oil are: (1) Absolute neutrality to prevent deterioration of metal; (2) freedom from resins or gums, which might clog passages when solidified by oxidation; (3) ability to remain liquid at 0° F.; (4) sufficient body to be retained in cylinders without excessive protection against leakage, and at same time low enough viscosity to flow freely. No vegetable oils answer above requirements; they contain too much acid, and thicken and solidify at too high temperatures. Animal oils have about same objections. Mineral oils are best; many of these are in the market. U. S. Government specifications for "Hydrolene," a standard product of many refineries, used for recoil cylinders of large guns, require: (1) Neutrality at ordinary temperatures and at 150° F.; (2) freedom from ash and saponifiable oil; (3) no trace of decomposition at 200° F.; (4) specific gravity of 0.835 to 0.87 at 60° F.; (5) cold test point (where flow ceases) to be below 0° F. Viscosity (by Saybolt Standard Universal Viscosimeter, used by Standard Oil Co.), 65 sec. plus or minus 10 sec. at 70° F., and preferably to vary as little as practicable from this between limits of 30° and 100° F., but not to be greater than 145 sec. at 30° F., nor less than 43 sec. at 100° F. (Viscosity is generally determined by time required for a given amount to flow through a standard orifice, as compared to water, sperm oil, or rapeseed oil.) Hydrolene will expand through heating. It will not freeze at 15° below zero. It discolors steel of guns after 12 months contact, but the metal is not otherwise attacked.

Leakage in Hydraulic Cylinders. Tests at Coffin Valve Co. on 52-in. hydraulic cylinders for 60-in. gate-valves, Catskill aqueduct (see Fig. 231) for leakage past the piston rings, consisted in placing a cylinder horizontal with the upper head removed; pressure was put behind the piston, which was prevented from moving. Under pressure of 50 lbs. per sq. in., leakage was 0.72 gal. per min.; 75 lbs., 0.97 gal.; 85 lbs., 0.86 gal. Reduction of leakage with increase of pressure from 75 to 85 lbs. appears to be due to pressure back of the rings forcing them out to make a tighter contact with the cylinder wall; 75 per cent. of the leakage came from the split in the outer ring; no special effort had been made to have a tight joint here; the space between abutting ends of the ring was 0.010 to 0.105 in. Some refinement in the joint design might



Size	A	B	C	D	E	F	G	H	J	K	L
4″	$11\frac{3}{4}$″	8.4 ″	$6\frac{3}{4}$″	$9\frac{1}{4}$″	$5\frac{1}{8}$″	$4\frac{9}{16}$″	$15\frac{3}{16}$″	$3\frac{1}{2}$″	9″	9″	$\frac{15}{16}$″
5″	$12\frac{1}{4}$″	9.55″	$7\frac{5}{8}$″	$10\frac{7}{8}$″	$6\frac{1}{2}$″	5″	17″	$3\frac{1}{2}$″	$9\frac{1}{4}$″	10″	$\frac{15}{16}$″
6″	$12\frac{1}{2}$″	10 7″	$7\frac{3}{4}$″	$11\frac{3}{4}$″	$7\frac{3}{8}$″	$5\frac{5}{8}$″	$18\frac{1}{2}$″	$3\frac{1}{2}$″	$9\frac{1}{2}$″	11″	1″
8″	$14\frac{1}{2}$″	13″	$8\frac{3}{4}$″	$14\frac{3}{8}$″	9″	$7\frac{7}{8}$″	$22\frac{3}{4}$″	4″	11″	$13\frac{1}{2}$″	$1\frac{1}{8}$″
10″	$15\frac{1}{4}$″	15 4″	$10\frac{7}{8}$″	$17\frac{5}{8}$″	12″	$8\frac{1}{2}$″	$26\frac{5}{8}$″	4″	$12\frac{1}{4}$″	16″	$1\frac{3}{16}$″
12″	$15\frac{3}{4}$″	17 7″	13″	$19\frac{5}{8}$″	$12\frac{7}{8}$″	$10\frac{5}{8}$″	$30\frac{1}{8}$″	4″	$13\frac{1}{4}$″	19″	$1\frac{1}{4}$″
14″	$17\frac{1}{2}$″	19 65″	$15\frac{3}{8}$″	$22\frac{1}{4}$″	$14\frac{3}{4}$″	$11\frac{3}{4}$″	$33\frac{1}{4}$″	4″	$16\frac{1}{2}$″	21″	$1\frac{3}{8}$″

Fig. 232.—Dimensions of Coffin type "F" gate valves.

reduce the leakage 30 per cent. That the piston rings were distended under pressure was shown by introducing a 0.004-in. feeler for an arc 6 in. long opposite the split, when under no pressure; under pressure the feeler could be pushed in only $\frac{1}{4}$ in. (width of ring, 1 in.). (See detail, Fig. 231.)

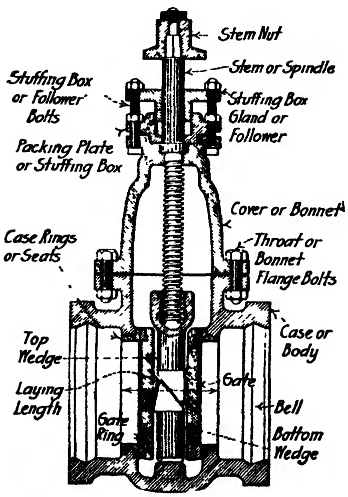


Fig. 233.
(Ludlow Valve Co.)

Friction in Hydraulic Cylinder. Tests were also made to find pressure to start the piston alone (no pressure on the plug). The cylinder was placed horizontally and the head of water measured in an attached standpipe; 3-ft. head moved the piston; 950 lbs. pressure moved an unbalanced load of 3500 lbs., giving a coefficient of starting friction of 0.27. The piston (spring) rings were designed to exert 4 lbs. pressure per sq. in. of cylinder walls. The test proved this. On such a basis, the total pressure necessary to move the piston would be 352 lbs., or a unit pressure of 0.17 lb. per sq. in. over the area of the piston.

Typical Double-gate Straightway Valve (Fig. 233). For this type the following claims are made: Principles of construction are such that gates cannot close until directly opposite the port or opening; gates are held rigid and

cannot get out of position, making it possible to open and close under any conditions; it is not possible for stem to bind in wedge; wear on faces of gates and seats is reduced to a minimum; operates equally well with pressure on either side of gates.

Limiting Number of Sizes of Valves. Gate valves of 6 sizes only are now used by New York City water department:

Pipe, inches	6	8	12 & 16	20, 24, 30	36 & 48	Above 48
Valve, inches	6	8	12	20	36	48

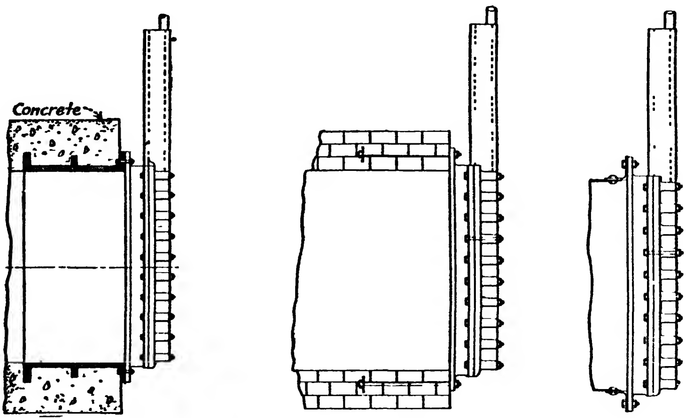


Fig. 234.—Methods of attaching sluice gates by flanges to masonry or pipe.

For designs and specifications, see E. N., May 27, 1915. Number of valves and repair parts necessarily kept in stock is much reduced. By using smaller valves in larger pipes shut-offs can be more quickly made and less room is occupied.

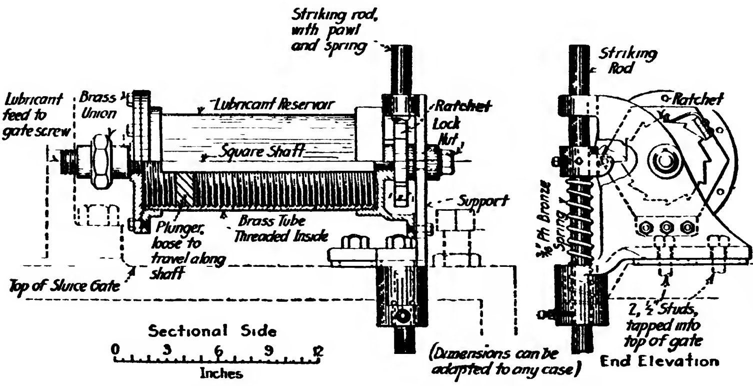
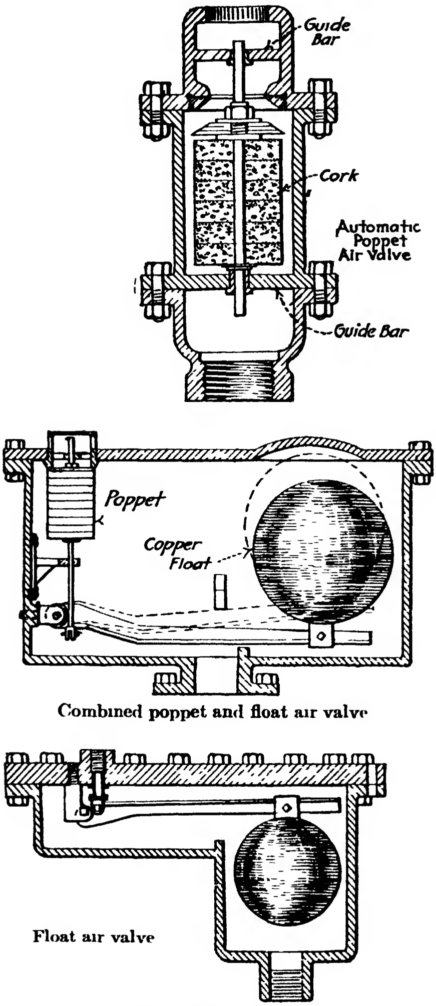


Fig. 235.—Automatic lubricator for submerged sluice gate screw. Charles river basin.
(E N , May 26, 1910)

Sluice Gates, Stock Sizes. Gates have not been standardized nor generally carried in stock. Makers, such as Coffin Valve Co., Boston; Chapman Valve Mfg. Co., Springfield, Mass.; Coldwell-Wilcox Co., Newburgh, N. Y.; Eddy

Valve Co., Waterford, N. Y.; and Hinman Hydraulic Mfg. Co., Denver, Col., have patterns for a wide variety of sizes of square, rectangular and circular gates, ranging from 10 × 10-in. to 216 × 87-in., and up to 108-in. diam. designed to meet various conditions as to pressure, etc. For small works selection can usually be made from these stock patterns; for large works it is sometimes necessary or economical to modify existing designs or prepare new ones. Rising stems should be used wherever practicable. Ball-bearing, roller-bearing and other friction-reducing types of operating stands or hoists are obtainable, also electric and hydraulic motor drives and hydraulic lifting cylinders.



Combined poppet and float air valve

Float air valve

Fig 236.—Eddy air valves.

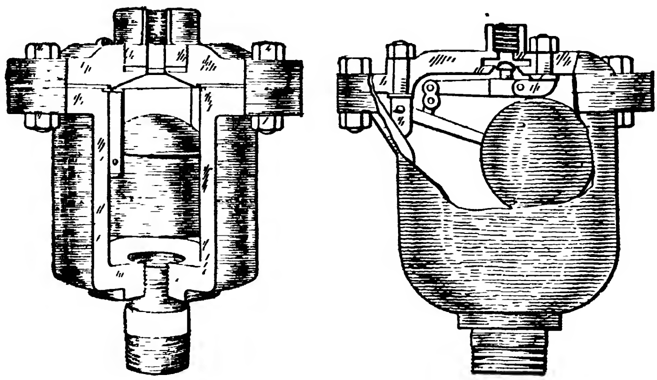
AIR VALVES*

Location. Air valves should be placed at summits of pipe lines, especially pipes with thin walls, to allow air to escape when pipe is being filled, and to enter, while emptying pipe. They help to render the pipe "fool-proof." The importance of air valves was shown in the failure of the steel pipe at Portland, Ore. (1911), under test (see page 315); air valves were not open. Sizes and locations of air valve can be determined only by computations and a study of the profile. The new 30-in. conduit for Syracuse, N. Y., is patroled every 48 hrs., at which time the 4-in. air valves are opened for the expulsion of air. It is found that the farther the water has flowed through the conduit, the less air is given off at summits.

Eddy Air Valves.† The *automatic poppet air valve* is drawn from its seat by vacuum caused when the pipe is being emptied; thus air enters the pipe and as it is filling, escapes until the valve is floated to its seat. These valves are not for use where air collects under pressure.

* See also p. 586.
† Made by Eddy Valve Co., Waterford, N. Y.

For this purpose the *combined poppet and float air valve* should be used. This air valve (Fig. 236) has an iron body, bronze mounted, and is a combination of the lever-and-float and poppet valve. The float is weighted and the length of its lever arm so proportioned that the combined leverage will give sufficient power to overcome the net upward pressure, against the poppet valve seat, of the gases that have accumulated in the chamber. The pressure of the air and gases will be the same as the water in the main at that point, and when the main is in service, the pressure will be more than sufficient to prevent an ordinary poppet valve from opening when it should do so. The combined valve is usually intended to have a 2-in. opening, yet it can have any required size, but when it is designed to act under the maximum pressure which can prevail in the main, it will be equally efficient under lesser pressures. Additional ordinary poppet valves, clustered in the same chamber, provide such an effective vent that they materially reduce the pressure. Two poppet valves, if for light pressures, can be operated by a single float. This valve will admit air in case



Automatic air and vacuum. Pressure.

FIG. 237.—Crispin air valves.

of a break in the main, which for steel pipe lines is a safeguard against collapse. In ordering, the conditions under which the valves are to be used, and the maximum pressure expected should be stated.

The *float air valve* (Fig. 236), is designed for use in the summits of pipe lines where air accumulates. When the air takes the place of the water, the float drops, carrying the valve away from the seat and permitting the air to escape. When the water enters the valve, the rising float causes the valve to close the air vent, preventing any more water escaping.

Air Valves Blown Shut. Great care should be exercised in the purchase of air and vacuum valves, that air can pass through at great velocity. Some valves can be blown shut with very little air pressure because the floats are so arranged that the escaping air from the main strikes them underneath, defeating the purpose. Floats and balls should be hard rubber, like meter disks. Fig. 236 shows three types of air valves.

Crispin Automatic Air and Vacuum Valve* operates as follows: The pipe

* Made by U S Metal & Mfg. Co., N. Y. City

line is tapped at the summit, and a nipple screwed in; the end of the nipple should be flush. The nipple carries a cylindrical casing. An opening in the cylinder head allows the escape of air. Suspended from the cylinder head is a brass basket in which rests a hollow brass ball. Sufficient clearance exists between the basket and the cylinder walls for the passage of air. As the air is exhausted, water enters the casing and floats the ball up against the seats, closing the space in the cylinder head; when air collects, or when the pipe is emptied, the ball drops back into the basket, being no longer floated, allowing the escape or entrance of air.

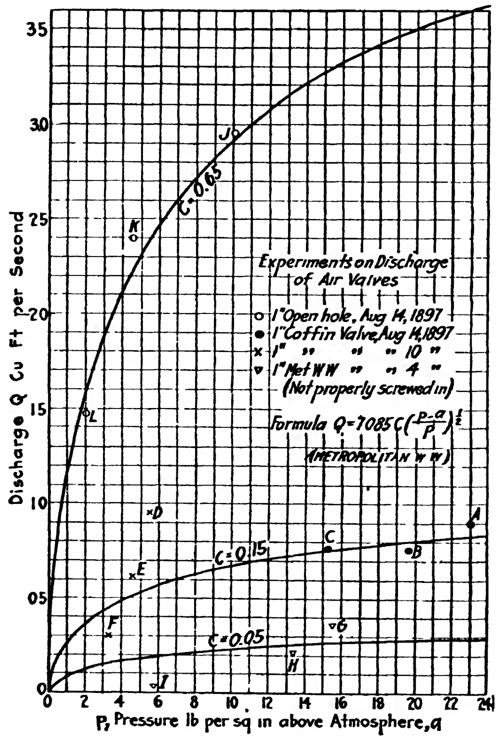


FIG. 238.

Crispin Pressure Air Valve (Fig. 237) is in standard sizes, $\frac{5}{16}$ to 1 in. diam. of air valve opening. It consists of a float chamber and a float attached to a lever, operating a small valve. With no air in the pipe, the float chamber is full of water. As air rises from the pipe, it displaces the water of the chamber, causing the float to drop, thereby opening an escape for the air. As the water returns, it lifts the float, automatically closing the valve.

PRESSURE-REGULATING VALVES

Pressure Regulation. The problem of satisfactory pressures is often complicated by topographical conditions; efficient fire pressure on high lands giving excessive domestic pressure in low lands. One solution is separate mains and a pumping station for each district; another is the use of pressure regulators. If a town is supplied from a source so high that direct pressure would be injurious to mains and services, pressure-regulating apparatus may be inserted in the supply main, or preferably on a by-pass, valved at either end, to provide proper domestic pressure. Such appliances are also used on old weakened mains to reduce high pressure to a safe pressure.

Pressure Regulators, to give satisfactory service, should be of simple design containing few parts and no delicate mechanism; should be strong, durable and reliable; all parts liable to rust should be made of non-corrodible material. Regulators should not close suddenly enough to cause water-ram

in the pipe lines. The apparatus should be operated entirely from the low-pressure side to avoid the effects of pressure fluctuations upstream. Valves too sensitively balanced operate under every pressure fluctuation and get considerable wear; ordinarily, operation under a pressure change of 2 to 4 lbs. gives satisfactory service. A pressure-regulator should always be located on a by-pass so that it can be cut out for repairs. In gritty water, a sand catcher just above a regulator chamber diminishes wear. Never place regulators at summits as the entrained air may interfere with their operation. Pressure-regulating valves cannot be used where the flow alternates in direction, as in a reservoir supply pipe which also serves as discharge. (A. O. Doane, Jl. N. E. W. W. Assn., 1906.)

Among manufacturers of pressure-regulating valves are: Ross Valve Manufacturing Co., Troy, N. Y.; Foster Engineering Co., Newark, N. J.; Golden-Anderson Valve Specialty Co., Pittsburg, Pa.; H. Mueller Mfg. Co., New York City; Waters Governor Co., Boston, Mass. (Doane valve). Simplex Valve and Meter Co., Philadelphia, Pa.; and Mason Regulator Co., Boston, Mass. The following notes were made in connection with 12-in. to 24-in. regulators, but apply to other sizes as well.

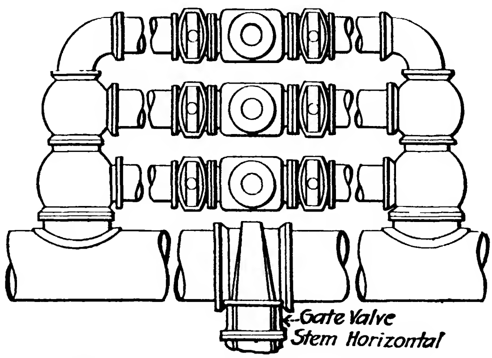


FIG. 239.—Battery of three regulators on large main, Worcester, Mass.
(Ross Valve Mfg Co)

If pressure regulators of very much too large capacity are installed, they are likely to be unsatisfactory, because working so near the closed position and with such small movements. For regulators of the Doane type, where it is desired to install large units to accommodate large flows in a comparatively near future but where for a few years flows will be small, regulators of the large sizes may be used fitted with diaphragms and spools, or plugs, of smaller size (see Fig. 248). At the proper time these latter can be readily removed without taking the regulator out of the pipe line and the full size parts installed with only a short interruption to service.

Like other automatic devices, pressure regulators need some attention, if they are to be kept in good condition; therefore, they should be inspected at suitable intervals by a competent mechanic. In a large system some regulators will have much harder service than others and should have more frequent attention.

Ross Pressure-regulating Valve (Fig. 240–241). A stem carries the pistons *E* and *F*, and disk valve *G*, which seats by an upward movement, on a leather collar. The areas of the pistons *G* and *F* are equal, preventing any tendency of the piston to move up and down. The actual reducing of pressure is done by valves *B* and *C*, which act on the check-valve principle; for instance, if the desired outlet pressure is exceeded, the upward pressure under piston *G* causes

an increase of pressure in the controlling chamber; this closes or diminishes the flow through the regulator *B* and opens the relief valve *C*, allowing the pistons to rise and close the valve. When the outlet pressure falls, the pressure in the chamber is reduced, the regulating valve opens and the relief valve closes, allowing the pistons to fall, thus opening the valve and increasing the pressure on the delivery side. The pressure at which the valve will open and close is determined by the hand wheels on *B* and *C*. This valve has few moving parts; no springs on the main valve; the seat, of the piston type, obviates chattering under irregular delivery when the valve is nearly closed; the valve is easily adjusted within its working limits, and has given satisfaction on the high-pressure fire service in New York and Brooklyn, holding the pressure with great uniformity and requiring the minimum of attention. The action of the valve is affected to a certain extent by the upstream pressure; should

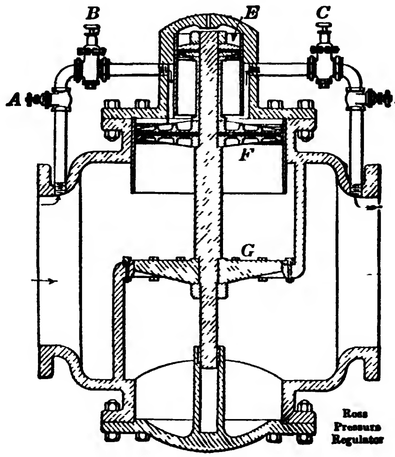


FIG. 240.
(Jl. N. E. W. W. Ass'n., 1906.)

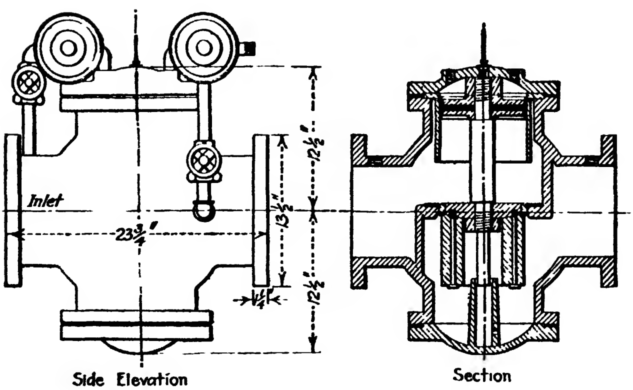


FIG. 241.—Eight-inch Ross regulating valve for stand pipe.
(Ross Valve Mfg. Co)

this fall much below the pressure for which the auxiliary valves are set, the main valve would either close or merely maintain reduced pressure on the downstream side; action is dependent on diaphragms and springs, introducing an element of maintenance and uncertainty, although experience shows but little trouble. The main valve seat and dash-pot plunger are leather- or rubber-packed; repacking necessitates cutting the valve out of service.

Table 121. Dimensions of Ross Pressure-regulator

Size of Pipe in	Distance between Flanges, in	Approximate Weight, lbs.
3	9½	180
4	11¼	220
5	11¼	250
6	17¾	380
8	23½	660
10	24½	870
12	30	1,300
14	37	1,550
16	37	1,675
20	47½	3,500
24	49	4,050

A. S. M. E. Standard Flanges (Table 100, p. 381.)

Foster Pressure Regulator (Fig. 242). Fluid is admitted in the direction indicated by the arrow (cast on the body), establishing an equilibrium between the balanced piston and the main valve (or clapper) and passes through to the outlet, and, by way of the port, to the piston chamber. When the fluid attains a pressure equal to the spring adjustment, it forces the piston upward, carrying the main valve toward its seat, and thus reducing the high initial to the desired terminal pressure. Any increase in the delivery pressure over the adjusted amount, acts upon and under the piston and gradually closes the main valve until the set pressure is again reached, while a drop or decrease in the delivery pressure acts *vice versa.* The main valve has a soft disk easily renewable. Turning the adjusting screw on top to the right increases, and to the left decreases, the delivery pressure. No other adjustment is necessary. This valve maintains constant terminal pressure without regard to volume. It

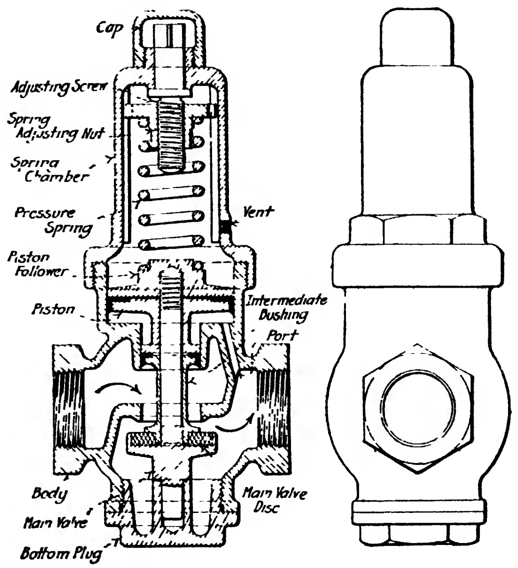
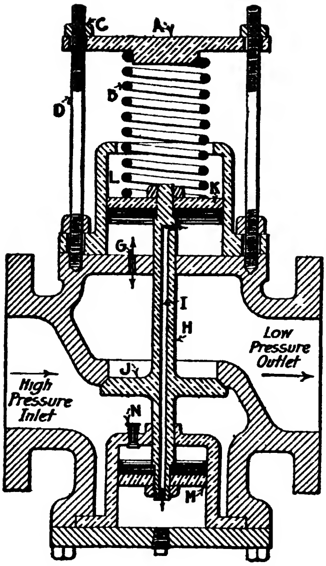


Fig. 242.—All bronze, except 2 piston cup leathers and rubber valve seats.

is capable of reducing as high as 300 lbs. to a delivery pressure of from 10 to 100 lbs. per sq. in. It can be installed in any position; is not affected by upstream pressure fluctuations, being controlled by the downstream pressure only; it is of the piston type, avoiding chattering under varying supply when nearly closed; seats are metallic, involving little renewal. The valves are spring-actuated, leaving them open to the chance of clogging by dust; spring action in one type is shortened by the introduction of levers. The downstream pressure actuates a diaphragm* which is liable to rupture from repeated bendings, although of small amplitude; renewal of the diaphragm necessitates cutting the valve out of line. There is no dash-pot* or other means of retarding "hunting"† should the delivery vary rapidly enough to cause fluctuations in the downstream pressure. These valves were used on some New York City work with good results.



Anderson Automatic Cushioned Water-Pressure-Reducing Valve (for high-pressure service, extra heavy, always cushioned in opening and closing) is shown in Fig 243 in closed position Spring *B* is adjusted by nuts *C* to required pressure Spring acts on large piston *K* which is attached to main valve spindle, forcing valve open Water on high-pressure side fills inlet chamber, opening auxiliary valve *N* exerting pressure on top of piston *M*, also acting as cushion Pressure also passes through port *G* underneath large piston *K* and through port *I* in valve spindle to underside of lower piston *M*, also acting as cushion When pressure on low side has reached pressure at which valve or spring *B* is set, pressure still increasing exerts a pressure under valve *J* and pistons *K* and *M* compressing spring *B* allowing valve *J* to close Pistons *K* and *M* prevent shock and chattering and cushion valve in opening and closing

FIG. 243.

Golden-Anderson Valve (Figs. 243–7) is also controlled by the downstream pressure, like the Foster valve; seats are of metal; the few parts are all of simple design; a dash-pot is provided to prevent "hunting." The valve is spring-actuated. The packing of rubber or leather necessitates cutting

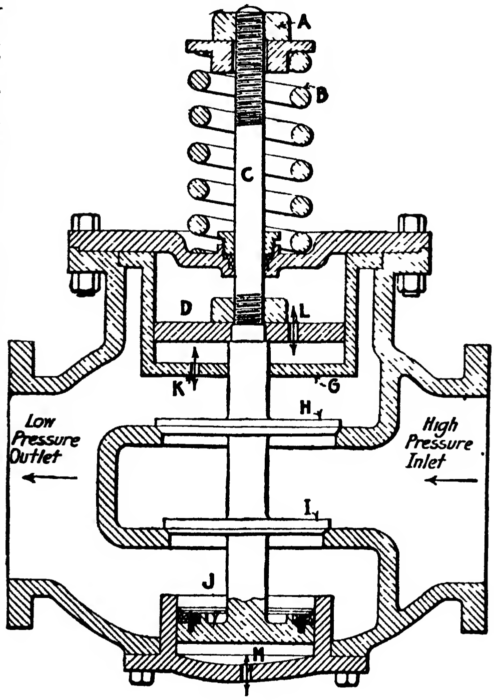


FIG. 244.

* The type to which these objections apply is not shown in Fig. 242.
† Hunting is vacillating of piston either side of pressure for which governing is set.

out the valve for repacking. The design of the seats renders chattering possible should the dash-pots prove ineffective.

Mueller Pressure-regulating Valve acts on the principle of a spring compressed by the pressure of the water on the delivery side. The principle of the Mueller Mfg. Co. is to use, for large-sized mains, a valve having about one-fourth the waterway area of the pipe line, with a reducer and increaser. Large regulators would lift from the seats so slightly under the minimum and average conditions of flow, as to cause excessive "wire drawing" of the seats,

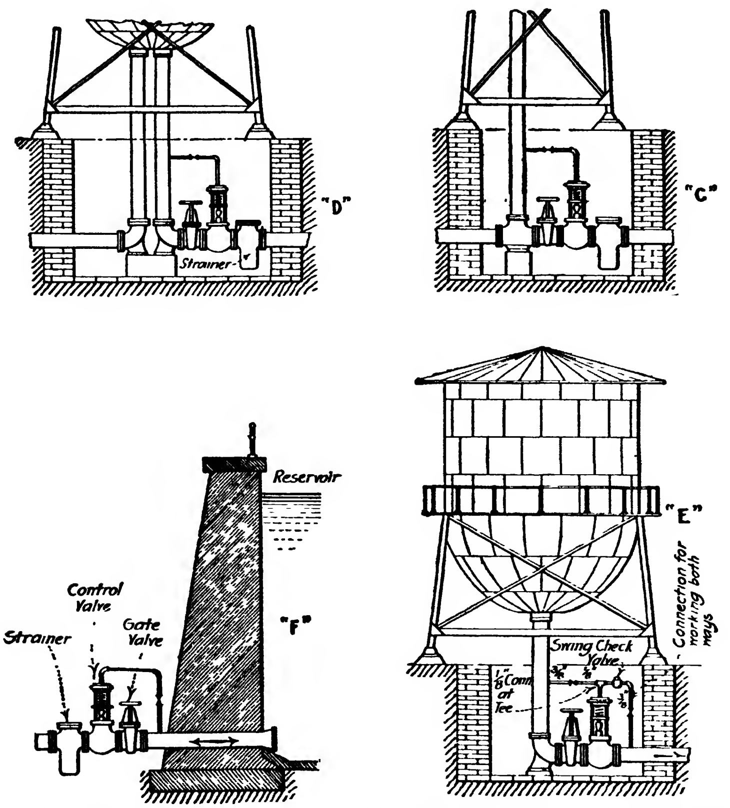


FIG. 245.--Anderson automatic controlling altitude valves attached to standpipes, tanks and reservoirs. (See page 439)

and render them easily obstructed and injured; they are also liable to chatter on the seats from pulsations in the line. For reducing pressures of 125 lbs. to 60 lbs. this firm guarantees a variation of not over 5 per cent. between minimum and normal delivery, nor more than 10 per cent. between maximum and normal. Each main valve must be equipped with a pilot valve of the same design, thus multiplying by two all dangers of breaks in springs and diaphragms; it also requires greater space in the valve chamber. A Mueller valve has been successfully used for some years on a standpipe in Brooklyn.

Anderson Automatic Controlling Altitude Valves (Fig. 247) shown in closed position. When water is drawn from tank, standpipe or reservoir, pressure is removed from top of diaphragm *R*, causing valve spindle *K* to rise, thus allowing high-pressure auxiliary valve *H* to close, and exhaust valve *Z* to open, permitting water above piston or valve *B* to escape through *M* and *N*. Removing pressure above valve *B* forces it to open. Water above valve acts as a perfect cushion in opening and at the same time that valve begins to travel upward, it draws in air through ports *F* in sides of body, to cushion the valve when closing. Water in the tank having reached required hight, pressure therefrom coming through pipe *T* distributes the pressure on top of diaphragm *R*, causing valve spindle *K* to close the auxiliary exhaust valve *Z* and to open the high-pressure auxiliary valve *H*, allowing the pressure to come on top of valve *B* through ports *M* and *L*. Owing to the greater area being on top of valve *B*, it is forced to its seat *E*, thereby shutting off all water. This action of the main valve *B* brings the air cushion into operation, compressing the air and forcing it out through port *F*, preventing shock in closing, The upper part of body is lined with bronze, also piston or valve is solid bronze fitted with rubber cups and disc, which prevents any wear of the liner or valve. Altitude valves are so constructed that electrical appliances can be quickly attached which will close the valves at any time by throw of an electric switch from pumping station or as many other places as desired. These valves will work both ways and maintain the water level in tanks, stand-pipes or

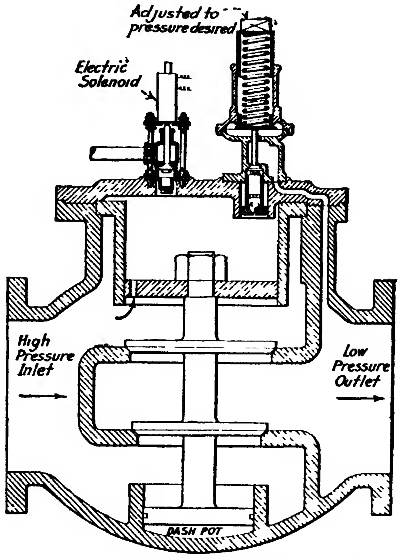
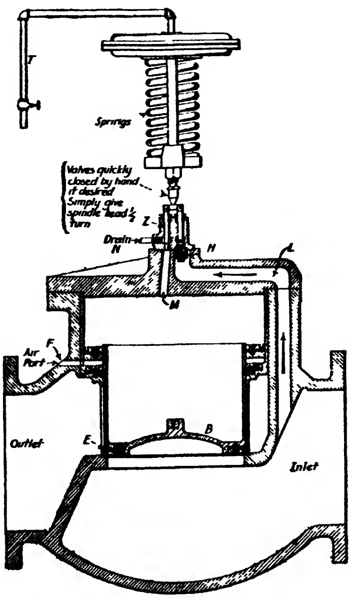


FIG. 246 —Anderson cushioned water-pressure-reducing valve (Quick opening by electrical operation).



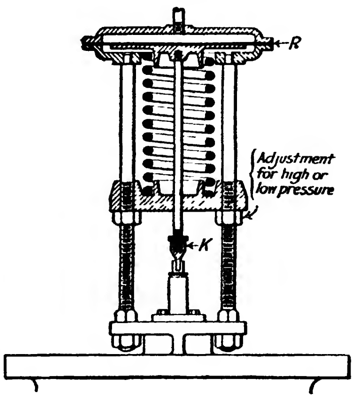


FIG. 247.—Anderson automatic controlling altitude valves.

reservoirs (Fig. 245) on a single supply and discharge line, and will allow water to flow back through the valves for distribution, whenever pressure is less on the inlet side of valve. Uniform stage of water is maintained. No floats or tank fixtures required. There is no overflowing in case pumps are put up to fire pressure in a system having low domestic pressure.

Doane Valve. Double-seated pistons are used; the pistons are connected by a cylinder instead of the usual rod, being conical and rounded off at each opening. The object of this is to produce considerable piston travel for small variations of flow; a slightly greater lift provides for large flow, as in the case of fire. This valve was adopted for the distribution system of Catskill aque-

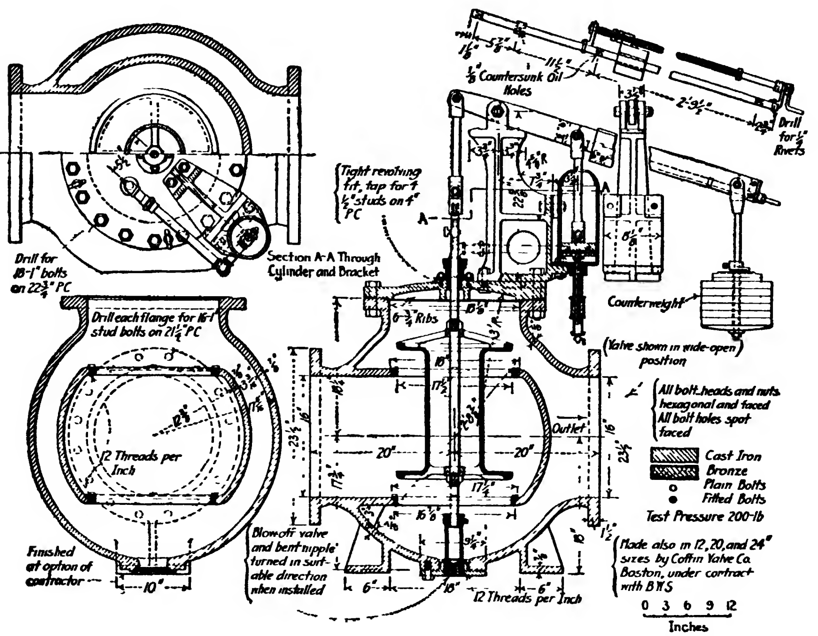


FIG. 248.—16-inch pressure regulating valve, Catskill aqueduct, New York City.

duct. Its advantages are extreme simplicity of parts; no springs; a dash-pot prevents "hunting" and is easily accessible for packing. The valve seats are metallic; the type allows chattering under varying delivery, but the port in the dash-pot cylinder is designed to prevent this.

Pressure-reduction and Regulation for Village Supplies. S. Whinery devised following scheme for reducing domestic pressure at Ellenville, N. Y. (gravity system), to 75 lbs., at same time leaving 115 lbs. pressure available for fires in elevated districts. On the 10-in. supply line a small reservoir or chamber was placed at a proper elevation to give 75 lbs. pressure in main part of town. Inlet was throttled by balanced float-valve. By-pass through the chamber gives full pressure for fire by opening a valve. Water is prevented from flowing back into the chamber by a check valve on the effluent pipe.—(E. R., Feb. 10, 1912, p. 149.) See Fig. 249, p. 440.

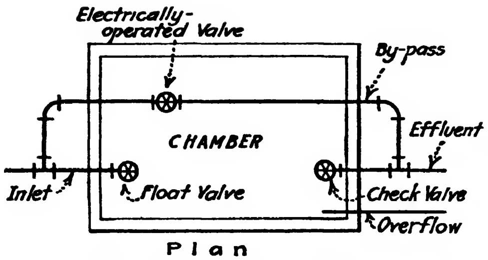
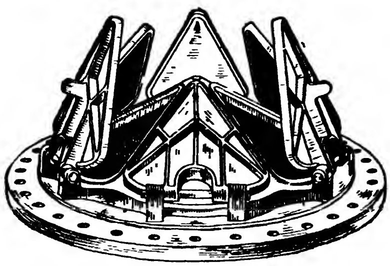


FIG. 249.—Ellenville pressure regulator.

Centrally Controlled Valves. It is sometimes desirable in small waterworks to have valves at remote reservoirs or tanks controllable from the superintendent's office or the pumping station. For long distances electric motors are the best means of operation; for shorter distances, hydraulic cylinders are also possible. For long distances electric transmission wires have numerous advantages over hydraulic pressure pipe. Valves thus operated from a distant point should have devices for showing at the point of operation the state of opening of each valve.

OTHER VALVE FORMS



Shell removed, showing triangular ports.

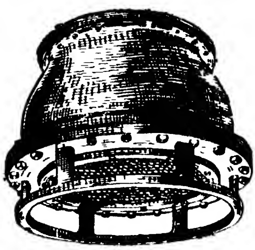


FIG. 250.

Coffin Foot Valve with Strainer. Without Springs, Bronze Mounted for Water. Casing and body, cast iron. Flaps or valves with seat facing and hinge pins, are solid bronze. Body consists of circular frame with triangular ports; aggregate area of openings is 15 per cent. in excess of area of connecting pipe. Frame is pyramidal and divided into multiple openings by reinforcing webs extending inward from flange. All openings converge toward apex.

Water being lifted raises all valves in unison. When valve is partially opened nearly entire weight is supported by pin upon which it hinges, and not upon column of water, as usual. A projection, or horn, upon each valve prevents its opening beyond a point of positive return. Bronze seat facing is cast in one piece and securely riveted in position. Both faces of valve and seat are machined and afterward scraped to water-tight joint. Copper wire strainers furnished when required.

If wire *strainers* be used on foot valves or in any similar positions, they must be of ample strength and very securely fastened in place. They must have very liberal excess of net open area over the cross-sectional area of the pipe or they will cause obstruction to the flow of water even when clean. Clogging of the screen not only obstructs the flow, but also may lead to such excessive pressure on the screen as to cause its failure, permitting fragments of the screen and the trash to be carried into the pump or other place where trouble will result. Screens should be examined and cleaned frequently.

Table 122. Coffin Foot Valve, Dimensions

(Flanges drilled A. S. M. E. Standard)

Sizes, in.	Extreme hight, in.	Greatest diam., in.	Number of valves (flaps)	Sizes, in.	Extreme hight, in.	Greatest diam., in.	Number of valves (flaps)
6	13¾	15½	1	20	26	36⅜	4
8	14 9/16	20¾	4	24	32⅛	44¼	5
10	15½	24	4	30	32 13/16	51	6
12	19	28	4	36	39½	58	6
14	19⅜	28⅝	4	42	51	74	6
16	20¾	30¼	4	48	59	84	8
18	23⅜	34¾	4				

Greatest diameter gives idea of size opening through which valve can pass

Table 123. Dimensions of Foot Valves and Screens

R. D. Wood & Co., Philadelphia, Pa

Size, in.	Foot valves				Foot valve screens	
	Flanges			Depth including rib, in.	Diam. of screen, in.	Depth to bottom, in.
	Pipe connection, in.	Bottom of screen, in.	Face to face, in.			
6	11	17½	8⅞	11⅞	13	5½
8	13½	20¾	10	13½	16	6½
10	16	23⅜	11 3/16	14 11/16	18	9
12	19	26	13 3/16	16 11/16	21½	10
14	21	30	15⅛	18⅝	25	12
16	23½	33½	17¼	20¾	27½	24½
18	25	36	19⅜	23⅜	31	16
20	27½	38½	21⅜	25⅜	33½	19

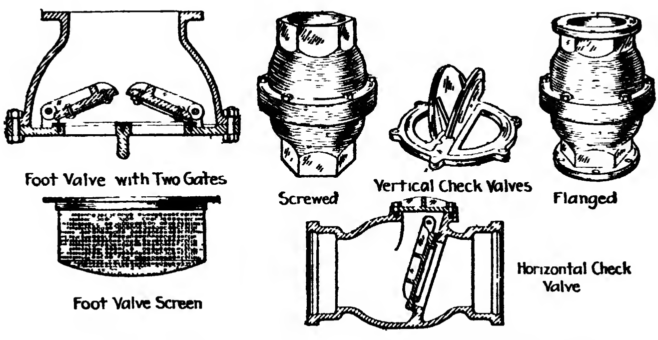


FIG. 251.—Foot valves and screens, and check valves.
(R. D. Wood & Co.)

Check Valves for Protecting Centrifugal Pumps. When a foot valve is used on suction pipe of a centrifugal pump pumping to great hight, or discharging through a long pipe, severe strain is put on the pump every time it is stopped, owing to "water-hammer," caused by suddenly closing the foot valve, which is often large enough to crack shell of pump. This trouble can best be prevented by placing a check valve immediately at the discharge flange of the pump. Large waterways must be provided through check valve.

Table 124. Dimensions of Check Valves, Bronze Mounted

R. D. Wood & Co., Philadelphia, Pa. (See Fig. 251.)

Size, in.	Hub		Flange	
	End to end, in.	Laying length, in.	Diam., in.	Face to face, in.
3	14¾	8¾	7½	10
4	18⅜	11⅜	9	12
5	19¾	11¾	10	13
6	23	15	11	15
8	27	19	13½	18
10	28½	20½	16	20
12	34	26	19	26
14	34	26	21	27
16	38	30	23½	30
18	39	31	25	32
20	43	35	27½	36
22	48	40	29½	40
24	54	46	32	44
30	68	59	38¾	56

Table 125. Dimensions of Coffin Standard Single-pivot and Double-pivot Flap Valves. (See Figs. 252 and 253.)

Spigot end					Double flange					Flange dimensions		
Size	A	B	C	D	E		F	G	H	I	J	K
4"	6 1/16"	5 7/16"	4 1/16	12"	8"	12"	3 1/16"	15/16"	9"	7½"	4	⅝"
6"	8¼"	7⅜"	5 5/16"	12"	8"	12"	4"	1"	11"	9½"	8	⅝"
8"	10½"	9 11/16"	6½"	12"	8"	12"	4"	1⅛"	13½"	11¾"	8	⅝"
10"	12¾"	12"	7¾"	12"	8"	12"	4¼"	1 3/16"	16"	14¼"	12	¾"
12"	15"	14¼"	9"	12"	8"	12'	4⅞"	1¼"	19"	17"	12	¾"
14"	17 5/16"	16 9/16"	10¼"	12"	8"	12"	5¼"	1⅜"	21"	18¾"	12	⅞"
16"	19⅝"	18⅝"	11 5/16"	12"	8"	12"	5¼"	1 7/16"	23½"	21¼"	16	⅞"
18"	21⅞"	20⅞"	12½"	12"	8"	12"	5½"	1 9/16"	25"	22¾"	16	1"
20"	23 15/16"	22 15/16"	13¾"	12"	8"	12"	6"	1 11/16"	27½"	25"	20	1"
24"	28"	27"	15¾"	12"	8"	12"	6⅛"	1⅞"	32"	29½"	20	1"
30"	34⅝"	33⅜"	19⅜"	14"	8"	12"	6⅜"	2⅛"	38¾"	36"	28	1⅛"
36"	41¼"	39¾"	23"	14"	8"	12"	7⅛"	2⅜"	46"	42¾"	32	1¼"

Note.—Dimensions *A*, *B* and *C* are same for all types. Dimensions *F*, *G* and *H* are same for single and double-flange types. *E* is made either 8" or 12", as ordered.

Anderson Automatic and Counterbalanced Valve (Anderson Mfg. Co. Pittsburgh, Pa.). Designed to meet demand for automatic water service valve in connection with standpipes and tank service adapted to varying pressures. Can also safely be connected direct to city mains without use of hydraulic relief valves. Body and top, extra heavy cast iron; upper portion of body lined with bronze; piston or valve is solid bronze, fitted with rubber cups and disk preventing metal parts coming in contact. Especially adapted for waterworks and railroads; cushioned at all times by air and water; will operate on ¾-to 1-in. variation. Used to maintain automatically uniform stage of water in troughs between tracks along railroads; can be quickly adjusted to fill ⅓, ½, ⅔ or full, as desired; also for various other purposes, maintaining any hight of water. (Fig. 254.)

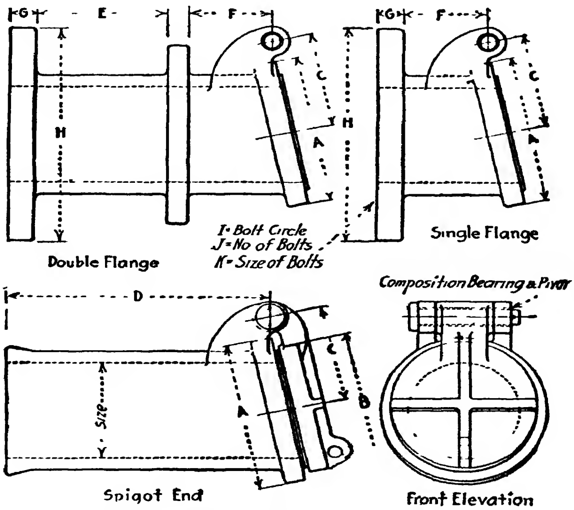


FIG. 252.—Coffin single-pivot flap valve.

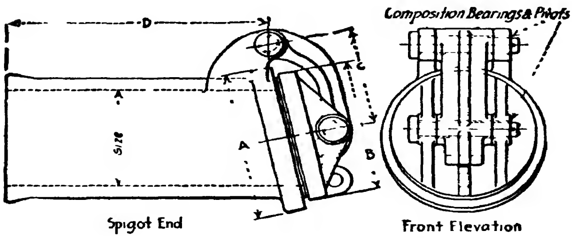


FIG. 253.—Coffin double-pivot flap valve.

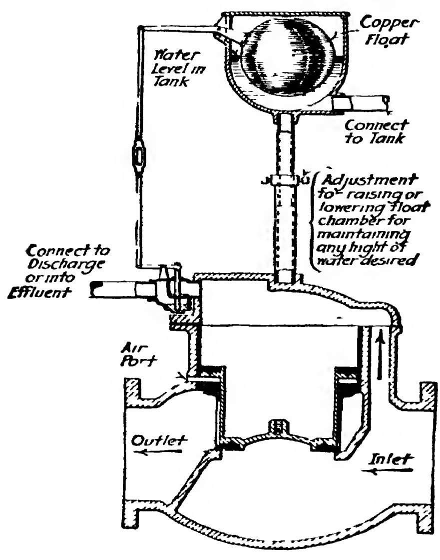


FIG. 254.—Anderson automatic and counterbalanced valve.

HYDRANTS

Standard Specifications for Post Hydrants, New England Waterworks Assoc.,* Apr. 15, 1914. 1. *Size.* *a.* Hydrants must† be designated by the number of 2½-in. hose outlets, for which they are designed. (If a steamer connection is also provided, the hydrant would be designated, for example, "two-way and steamer connection.")

b. The net area of the waterway at the smallest part other than at the valve opening when the hydrant is wide open must not be less than 120 per cent. that of the valve opening, and there must be sufficient clear waterway through the hydrant when wide open to allow the passage of a ball at least 1⅞ in. in diam. for a two-way and 2⅜ in. for three- and four-way hydrants. (In new designs it is recommended that inside diam., especially at outlets, be 7 in. for two-way and 8 in. for three- and four-way.)

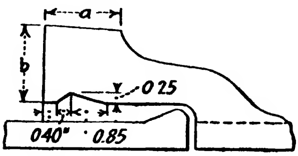


Fig. 255.—Standard bell-end for hydrant (6-in. and 8-in.).

c. Hydrants must be fitted with bell ends to fit standard cast-iron pipe, the dimensions of which are given in Table 126, or with flanges of standard dimensions and having standard bolt layouts, as given in Table 127. Holes are not to be drilled on the center line, but symmetrically each side of it.

Table 126. Post Hydrant Bells. N. E .W. W. Assoc.

Nominal pipe diam., in	Classes	Actual outside diam of pipe spigot, in.	Pipe sockets		"a"	"b"
			Diam., in	Depth, in		
6	All	7 10	7 90	3 00	1 50	1 40
8	All	9 30	10 10	3 50	1 50	1 50

Table 127. Post Hydrant Flanges. N. E. W. W. Assoc.

Size of pipe, in	Diam of flange, in	Flange thickness at edge, in	Diam of bolt circle, in	Number of bolts	Size of bolts, in.
6	11	1	9½	8	¾ × 3
8	13½	1⅛	11¾	8	¾ × 3½

Where working pressure is from 150 to 250 lbs, the standard for heavy flanges must be used, see p 381.

2. *General Design.* *a.* Hydrant may be of compression or gate type.

b. Hydrants must be designed to withstand safely a working pressure of 150 lbs., with a factor of safety of at least 5 at the working pressure. For example, a hydrant whose working pressure is 150 lbs. must resist a pressure of 750 lbs. before breaking.

c. Valve when shut must remain reasonably tight when upper portion of barrel is broken off.

d. Any changes in diam. of the water passage through the hydrant must have easy curves, and all outlets must have rounded corners of good radius.

e. With the hydrant discharging 250 gals. per min. through each 2½-in. hose outlet, the total friction loss of the hydrant must not exceed 1¾ lbs. for two-way, 2¼

* Similar specifications issued by Am W W. Ass'n, 1913. Space prevents discussion of differences
† "Shall" is preferable to "must" throughout.

lbs. for three-way, and 3 lbs. for four-way hydrants, except when fitted with inside hose gate valves, in which cases the loss must not exceed $2\frac{3}{4}$ lbs. for three-way and $3\frac{1}{2}$ lbs. for four-way hydrants.

f. Hydrants must be so designed that with extraordinary usage they will not cause serious water-hammer.

g. Hydrants must be fitted with two lugs so that the leaded joint underground can be strapped.

h. When hydrant barrel is made in two sections, the upper flange connection must be at least 2 in. above the ground line.

In clayey soils where the ground packs closely about the hydrant and tends to grip it, freezing and consequent heaving of the ground would bring some strain on the hydrant, especially if there were flanges near the ground level. To overcome this effect the flanges should be put above ground and the hydrant surrounded from bottom to ground level with a 3-in. layer of gravel. About $\frac{1}{2}$ bu. of small stones should be placed around the base of every hydrant to serve as a drain, unless the drip is connected to some waste pipe.

3. *Material of Body*. *a*. The hydrant body must be made of cast iron.

4. *Materials*. *a*. Iron castings same as cast-iron pipes. (See page 450.)

b. All wrought iron or mild steel used must be of the best quality of double refined iron, of a tensile strength of at least 50,000 lbs. per sq. in.

c. All composition or other non-corrodible metal used must be of the best quality, have a tensile strength of not less than 30,000 lbs. per sq. in., with 5 per cent. elongation in 8 diam. and 5 per cent. reduction of area at breaking point. Composition for inside hose-valve stems must have a tensile strength of not less than 55,000 lbs. per sq. in. and an elastic limit of not less than one-half the tensile strength.

5. *Hose Valves and Nipples*. *a*. Hydrants must have at least two hose connections.

b. If hose-gate valves are used they must be of the outside detachable type or be built inside the barrel. The outside hose-gate valves must be made of composition or of iron with composition trimmings, with lugs cast on the valve body, and each valve must be bolted to the hydrant by two $\frac{3}{4}$-in. tap bolts, spaced horizontally $5\frac{5}{8}$ in. on centers. The valves must not project farther than necessary, and must be of the inside screw type, placed in a vertical position, with the hand-wheel at least 3 in. below the base of the operating nut.

Inside hose-gate valves must have composition metal working parts and be of rugged design, and must not introduce an unnecessary friction loss. There must be ample clearance between the gate and the hydrant body when the gate is in any position. The gate and parts should be interchangeable, and the valves should be located so as to be as accessible as possible for repairs. The gate must be designed so that it cannot come off in use. The top of the stem must be below the level of the hydrant stem nut, so that the hydrant wrench can be freely operated.

c. Hose nipples must be of composition metal threaded with a fine thread into the hydrant, and securely pinned in place. If desired, the nipple may be cast with two side lugs and be bolted to the hydrant.

d. Hose threads on all hydrants to be installed in any given district must of necessity be interchangeable with those already in service, but, where practicable, threads should conform to the National Standard. The essential features of the "National Standard" thread in the $2\frac{1}{2}$-in. size are a 60-degree V-thread, outside diam. on male threads of $3\frac{1}{16}$ in., and $7\frac{1}{2}$ threads per in.

e. The stems of the hose valves must be not less than $\frac{3}{4}$ in. diam. for the $2\frac{1}{2}$-in. valves, and not less than $\frac{7}{8}$ in. diam. for the valves at the steamer connections.

f. The stem nuts of all inside hose and steamer connection gate valves must be ½ in. square.

6. *Hydrant Valve*. *a*. The valve seat must be made of composition metal securely threaded in place.

b. The valve must be faced with a yielding material, such as rubber or leather, except that if of the gate type, a bronze ring may be used. The valve must be designed so that it can be easily removed for repairs without digging up the hydrant.

c. The clearance of parts must be such that corrosion will not make the parts inoperative.

7. *Drip Valve*. *a*. A positively operating non-corrodible drip valve must be provided and arranged so as to drain the hydrant when the main valve is shut.

b. The seat of the drip valve must be made of non-corrodible metal and must be securely fastened in the hydrant. All other parts of the drip mechanism must be designed to be easily removed without digging up the hydrant.

8. *Operating Stem*.* *a*. The threaded section of the operating stem when located in the waterway must be of composition or other non-corrodible metal, but when above the stuffing box it may be of wrought iron. The operating stem where it passes through the stuffing box and gland, must be of composition metal or be fitted with a composition metal covering. The diam. of the operating stem at base of thread must be not less than 1⅛ in. for gate type of hydrant and 1 in. for compression and toggle types. The remainder of stem may be of iron and the diam. must be not less than 1⅜ in. for gate type of hydrant and 1¼ in. for compression and toggle types. The operating stem must be attached so that in operation it will be impossible for it to become detached.

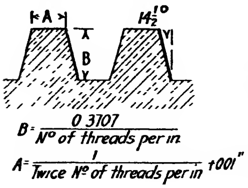


FIG. 256.

b. The stem must terminate at the top in a nut of pentagonal shape, finished with slight taper to 1½ in. from point to flat, except for hydrants to be installed where existing hydrants have different shape or size of nut, in which case the additional hydrant must have the same operating nut as the old ones for uniformity. The nut socket in the wrench must be made without taper so as to be reversible.

c. The thread which operates the valve must be Acme, half V or square. The Acme standard thread is shown in Fig 256.

9. *Stuffing Box and Gland*. *a*. The stuffing box and gland must be of composition metal or bushed with it. If a packing nut is used, it must be of composition metal. The bottom of the box and end of the gland or packing nut must be slightly beveled.

b. Gland bolts or studs must be of composition metal, wrought iron, or steel, at least ½ in. diam. Nuts must always be of composition metal.

10. *Hydrant Top*. *a*. The hydrant top must be designed so as to make the hydrant as weatherproof as possible, and thus overcome the danger of water getting in and freezing around the stem. Provision must be made for oiling both for lubrication and to prevent corrosion. A reasonably tight fit should be made around stems.

b. There must be cast on the hydrant top, in characters raised ⅛ in., an arrow at least 2½ in. long, showing direction to open, and the word "OPEN" in letters ¾ in. high.

11. *Hose Caps*. *a*. Hose caps must be provided for all hose outlets, and must be

* The iron threaded section is permissible only where the hydrants are to receive the best of care and be kept oiled, otherwise composition must be used to insure reliable operation

securely chained to the barrel with a welded chain of wire, not less than $\frac{1}{8}$ in. in diam.

b. The hose cap nut must be of the same size and shape as the operating nut.

c. A leather or rubber washer must be provided in the hose cap, set in a groove to prevent its falling out when the cap is removed. If desired, a lead washer or disk may be used so retained in the cap that it will not fall out.

12. *Marking.* *a.* Hydrants must be marked with the name or trade mark of the manufacturer and the year of manufacture. All letters and figures must be cast on the hydrant well above the ground line. They must be 1 in. high and raised $\frac{1}{8}$ in. on the casting.

13. *Testing.* *a.* Hydrants, after being assembled, must be tested to at least 300 lbs. per sq. in. before leaving the factory. If the working pressure is over 150 lbs. per sq. in., the hydrants must be tested to twice the working pressure. The test should be made with the valve open in order to test the whole barrel for porosity, and strength of hydrant body. A second test should be made with the valve shut in order to test the strength and tightness of the valve.

b. Hydrants must be fully opened and closed before shipping in order to test the freedom and strength of the parts. The conditions of the test should be made as severe as are liable to occur in service when using a hydrant wrench at least 17 in. long.

14. *Direction to Open.* *a.* All hydrants must open to the left (counter-clockwise), except where existing hydrants open to the right, in which case additional hydrants should turn the same as the old ones for the sake of uniformity.

Hydrant Spacing. The Committee of the Am. W. W. Assn. recommended smaller standard sizes of hydrants at frequent intervals, rather than large hydrants with numerous outlets at longer intervals. Hydrant spacing should be governed largely by the character and value of the property protected. In thickly built mercantile and manufacturing sections, hydrants should be about 200 ft. apart, and not more than 500 or 600 ft apart at the maximum anywhere. In other words, it should be possible to concentrate a sufficient number of effective streams on any building which may burn, and for this end lengths of hose should not be greater than 250 to 350 ft.

"Standard" Sliding Frost Case Compression Fire Hydrant (The A. P. Smith Mfg. Co., Newark, N. J.). Fig. 257. In use since 1870. Principal working parts are bronze: opening nut, holding down nut, packing box gland, nozzles, waste valve, combined waste-valve chamber and valve seat, and valve lock nuts. Steel spindle is bushed with brass where it passes through packing. All parts are extra heavy, true to gage and interchangeable. Standpipe is in two sections with twice as many bolt holes in lower as in upper; position of nozzles in relation to street can therefore be changed as desired without digging down to shoe or altering gasket. Leather gasket in base joint is screwed on the standpipe, so that if it should be necesary to remove it the gasket would come out with it, and could therefore be replaced without difficulty. Drip hole is situated so as to drain all water above the valve from the standpipe.

O'Brien Fire Hydrant (The A. P. Smith Mfg. Co., Newark, N. J.). Designed to meet demand for compression hydrant that can be repaired without removing standpipe. Drip valve is bronze, faced with rubber. Hydrant valve is bronze, faced with hard-rolled leather. Opening nut, holding down nut, upper section of spindle, hose nozzles, valve seat, drip valve, socket and guide,

and lock nuts are bronze. By removing the hydrant bonnet and top, the drip valve, drip-valve socket and all working parts can be removed and replaced without disturbing the standpipe. In opening a hydrant, the drip valve, which drains all water from the standpipe, is positively shut before the main valve leaves its seat; and in closing, the main valve is seated before the drip valve begins to open. Standpipe is tapered and therefore cannot be heaved by frost. All parts are heavy, true to gages and interchangeable.

Hydrant Double-nozzle Shut-off. To allow a second line of hose to be connected to a hydrant while one nozzle is in service, without shutting off

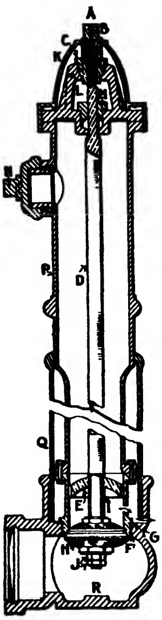


FIG. 257. "Standard"

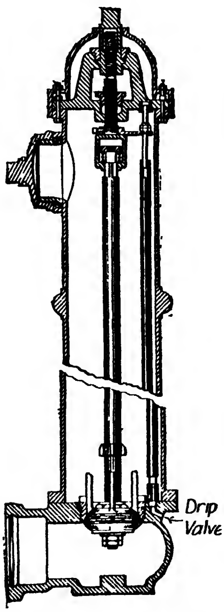


FIG. 258.—O'Brien fire hydrant.

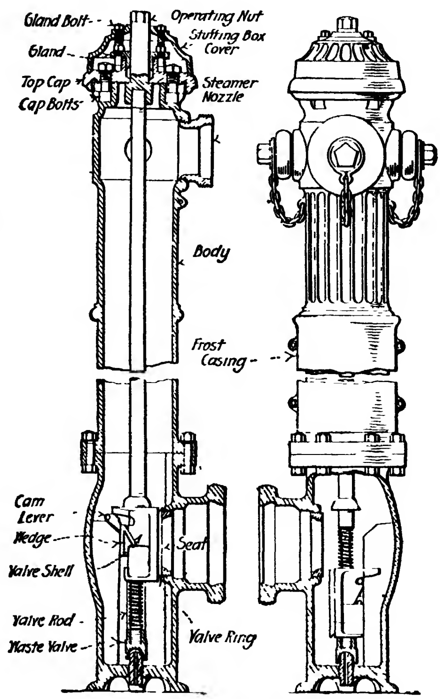


FIG. 259.—Walker rubber-faced fire hydrant.

from the main, The A. P. Smith Co. furnishes hydrants to order with a double nozzle shut-off. It is claimed to be superior to any form of independent nozzle gate, as when the hydrant is in service it does not obstruct the flow; and owing to the principle of construction the operating spindle can be and is made as strong as the hydrant spindle and operated by the same wrench.

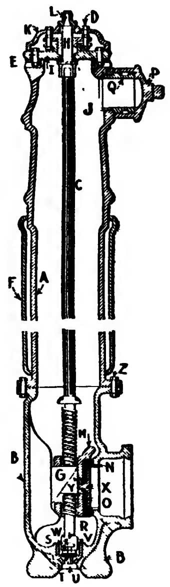
Walker Rubber-faced Fire Hydrant. Valve has removable rubber face. Sediment between valve faces will not scratch and cause leaks. There are no rubbing or scraping valve faces; action is positive. There are two distinct motions of the valve in opening: first, away from the seat horizontally, then down out of the waterway, reversed when closing. Inexperienced workmen cannot twist or bend the rod, because all strain is tensile, between points only a

few inches apart. No digging is needed to remove the valve and waste, as valve, rod and waste are all in one section—a simple lift brings all parts out at top. Top cap, secured by three bolts, is easily removed. Waste device is made of hard bronze in the form of a ball joint; there are no rubber or leather parts to cause trouble; it cannot be damaged by sediment. No flooding can result from a broken post; when closed, the valve is locked upon its seat. Should the standpipe be broken by accident, the hydrant still remains tight. All top parts can be removed safely without shutting off water from the main. (Norwood Engg. Co.)

Table 128. Ludlow Fire Hydrant (Bronze Mounted; Rubber-faced Gate)

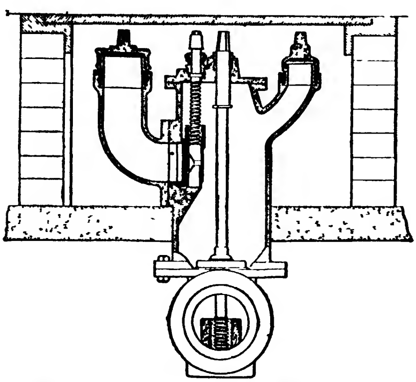
Diameter of seat ring or valve opening, in	Inside diameter of standpipe or hydrant barrel, in	Hub end, bottom connection, size connecting pipe, in
2	3	2
3	$4\frac{5}{8}$	3 or 4
4	$5\frac{3}{4}$	3 or 4
4	$5\frac{3}{4}$	6
$4\frac{1}{2}$	$6\frac{1}{4}$	6
5	7	6
6	8	6

Tested at 300 lb. per sq in., water pressure.

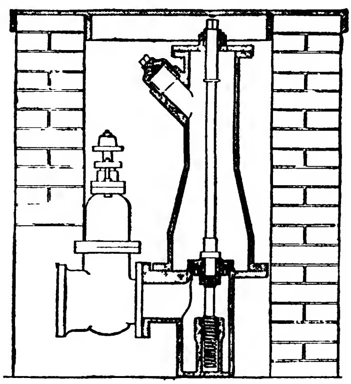


A, barrel or stand pipe, *B*, bottom, *C*, stem; *D*, follower or dome bolts, *E*, packing plate bolts; *F*, frost case, *G*, bronze wedge nut; *H*, bronze sleeve, *I*, packing plate, *J*, packing gland or follower; *K*, dome or cover; *L*, top nut, *M*, gate, *N*, bronze seat ring, *O*, gate rubber, *P*, nozzle cap, *Q*, bronze nozzle, *R*, bronze screwed end drip piece; *S*, bronze drip washer; *T*, drip rubber, *U*, bronze drip nut; *V*, bronze drip cup, *W*, bronze drip bolt, *X*, bronze gate plate, *Y*, bronze gate plate nut, *Z*, flange bolts.

Fig. 260.—Ludlow fire hydrant.



WITH ONE HOSE AND ONE STEAMER NOZZLE.



WITH AUXILIARY VALVE ATTACHED TO HYDRANT.

Fig. 261.—Clow tropical flush hydrants.

Clow Tropical Flush Hydrants are constructed for warm climates or where pipes are drained during the winter. They are provided with any size inlet,

either threaded, flanged or hub ends, and as many nozzles, either steamer or hose, as required.—(Made by Jas. B. Clow & Sons, Chicago.)

Lubricant. Waterproof graphite grease applied to the valves and caps of fire hydrants forms a coating that cannot be washed off even by the rushing stream of water; it preserves the valves and cap from rust, and from sticking by "freezing." It will not gum or become rancid.

When Ordering Hydrants give: Kind wanted; size of valve opening; depth from surface of ground to bottom of connecting pipe; size of connecting pipe; whether screw, bell or flange end, number and size of nozzles. Send a nozzle cap as gage for thread and nut, or give exact outside diam. and kind of nozzle threads, also size and form of nut; state whether to open to right or left.

Quality of Iron for Pipes, Special Castings, Valves and Hydrants.* Cast iron shall be of good quality, and of such character as shall make the metal of the castings strong, tough, and of even grain, and soft enough to satisfactorily admit of drilling and cutting. The metal shall be made without any admixture of cinder iron or other inferior metal, and shall be remelted in a cupola or air furnace.

Tests of Material—Specimen bars of the metal used, each being 26 in. long by 2 in. wide and 1 in. thick, shall be made without charge as often as the engineer may direct, and in default of definite instructions the contractor shall make and test at least one bar from each heat or run of metal. The bars, when placed flatwise upon supports 24 in. apart and loaded in the center, shall for pipes 12 in. or less in diameter support a load of 1,900 lb. and show a deflection of not less than .30 in. before breaking, and for pipes of sizes larger than 12 in. shall support a load of 2,000 lb. and show a deflection of not less than .32 in. (shall support a load of 2000 lb. and show a deflection of not less than .30 in. before breaking, or if preferred, tensile bars shall be made which will show a breaking point of not less than 20,000 lb. per sq in).† The contractor shall have the right to make and break three bars from each heat or run of metal, and the test shall be based upon the average results of the three bars. Should the dimensions of the bars differ from those above given, a proper allowance therefor shall be made in the results of the tests

*N E W W. Ass'n Specifications.

† Preceding clause in brackets shows difference between Am W. W Ass'n and N E W W Ass'n Specifications

CHAPTER XXI

SERVICE METERS

VENTURI METERS

Description and Hydraulic Principles. Venturi meter consists of two parts, the tube and the instrument, and has advantage of dispensing with all mechanism in path of flowing water, readings depend solely upon relative pressures in two small pipes leading from pressure chambers at upstream end and throat of tube to instrument. (See Fig. 262.) Principle is based upon relation between velocity and pressure in water flowing through a continuous pipe of varying cross-sectional area. This principle was first stated in 1797 by J. B. Venturi, an Italian philosopher, who had observed that fluids discharging through an expanding nozzle exert a sucking action at the small diam. which diminishes as diam. increases toward outlet. In 1887, Clemens Herschel, after elaborate experiments, adapted the principle to accurate measurement of fluids by means of the Venturi meter It is now used for water, oil, steam, gas

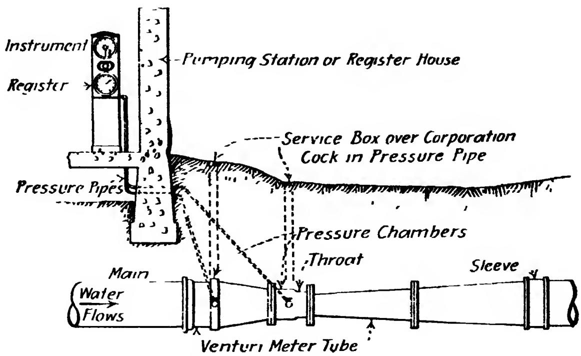


FIG 262 --Typical arrangement of Venturi meter for gravity or pump discharge main

and other fluids. In passing from full section to throat, velocity is gained at expense of pressure. Pressure at throat is thus less than at upstream entrance; water passing through a tapering tube has been found by experiment to be nearly equal to the theoretical discharge from the throat opening at a velocity corresponding to difference of pressure between throat and entrance. Tubes are usually cast iron, but for special cases may be of steel plate, reinforced concrete, wooden staves or other material. There are several kinds of instruments, some indicating the differences of pressures at entrance and throat, others registering or recording the flow in various units; some combine indicator, register and chart recorder. For determining the total quantity for the period covered by a dial chart, or any fraction of such period, the Builders Iron Foundry furnishes a special planimeter. (See also Fig. 362, page 706.)

Losses through a Venturi Meter. Venturi meter tubes do not introduce appreciable friction. Usually the normal rate of discharge through the pipe line is about one-half the measuring capacity of the meter. At this rate increased friction due to the meter tube is only $\frac{1}{4}$ lb. per sq. in. Even when the meter is operated at its highest capacity the friction amounts to only 1 lb. per sq. in. The most advantageous divergence for the downstream tube, and probably for any diverging tube, to obtain the best effect in developing Venturi action, is 1 in. increase in diam. for each $11\frac{1}{2}$-in. increase in length, equivalent to an angle at the vertex of the cone of a trifle more than 4° 58′.

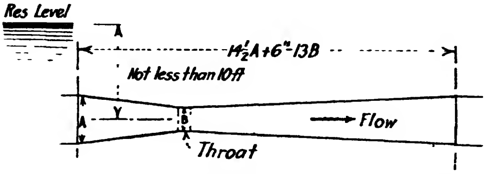


FIG. 263.—Relation of length and diameter of Venturi meter.

Diameter of Throat may vary from $\frac{1}{3}$ to $\frac{1}{2}$ diam. of upstream end, to be within limits determined by reliable experiments. The limit of throat velocity usually adopted is 38 ft. per sec. Where a sufficient ratio between the diam. of the upstream end and the throat cannot be obtained in the ordinary size of pipe, the pipe may be increased in diam. for the approach to the meter, Fig. 264.

FIG. 264.

Directions for Installing Venturi Meter Tubes.* Meter tubes are set in pipe lines in same manner as ordinary pipe, the shorter cone forming the inlet, or upstream end. A notch in the edge of each flange denotes the top. It is not essential that the tube be horizontal; it can be inclined or vertical. A straight length of pipe of same diam. as meter tube should immediately precede the *inlet* and contain no gate valve or other fitting liable to disturb the smooth flow of the water. The length of this pipe should be at least six times diam. of tube for sizes up to 24 in. and at least 12 ft. for larger sizes. If *outlet* end of tube is of different diam. from pipe line, an increaser or decreaser should be placed at this point. It is unnecessary to have a straight length of pipe on the outlet. For standard installations both tube and instrument should be set where the working pressure is at least 12 lbs. per sq. in. Frequently, however, this requirement may be modified; a tube may be set with its center line only 10 ft. below hydraulic gradient. Two small pressure pipes connect meter tube with instrument. These may be brass, lead, lead-lined iron or other non-corrodible piping, $\frac{3}{4}$ in. diam. if length is 50 ft.; 1 in. diam. if length is 100 ft., etc., and connection should be made at the *side* of each pressure chamber. The piping should have a pronounced up or down grade, contain no summits nor depressions where air or silt might collect, and a valve (or corporation cock) should be placed on each pressure pipe close to tube. If a summit or depression is absolutely unavoidable, a blow-off valve should be provided at such point. All joints must be perfectly tight and piping properly protected from frost.

* Builders Iron Foundry, Providence, R. I.

Table 136. Standard Venturi Meter Tubes

Capacities, Lengths and Weights

Inlet and outlet diam., in.	Length* Ft. In	Measuring capacity, thousand gal. per 24 hrs. Min.	Max	Approx. weight, lbs.
2	1 11⅞	4	51	50
	1 10¼	6	73	
	1 7	10	130	
2½	2 4⅝	7	100	85
	2 3	10	130	
	1 11¾	16	203	
3	2 11	10	130	110
	2 7¾	16	203	
	2 4½	23	293	
4	4 3¾	16	293	160
	3 10⅞	26	343	
	3 6	40	520	
5	5 1⅜	26	343	275
	4 8½	40	520	
	4 2	63	813	
6	5 11	40	520	450
	5 4½	63	813	
	4 10	90	1,170	
8	7 6¼	76	983	700
	6 11¾	106	1,373	
	6 2	160	2,080	
10	9 4¾	106	1,373	1,100
	8 7	160	2,080	
	7 6	250	3,250	
12	11 0	160	2,080	1,550
	9 11	250	3,250	
	8 10	360	4,680	
14	12 10½	203	2,633	2,200
	11 6¼	331	4,298	
	10 2	490	6,370	
16	14 5¾	276	3,583	3,000
	13 1½	423	5,493	
	11 6	640	8,320	
18	16 1	360	4,680	3,700
	14 5½	563	7,313	
	12 10	810	10,530	
20	17 11½	423	5,493	4,750
	16 4	640	8,320	
	14 2	1,000	13,000	
22	19 10	490	6,370	5,700
	17 8	810	10,530	
	15 6	1,210	15,730	
24	21 2	640	8,320	6,800
	19 0	1,000	13,000	
	16 10	1,440	18,720	
26	23 0½	723	9,393	8,300
	20 4	1,210	15,730	
	18 2	1,690	21,970	
28	24 11	810	10,530	9,600
	22 2½	1,323	17,193	
	19 6	1,960	25,480	
30	26 3	1,000	13,000	11,000
	23 0	1,690	21,970	
	20 10	2,250	29,250	
32	28 1½	1,103	14,333	12,700
	25 5	1,690	21,970	
	22 2	2,560	33,280	
34	30 0	1,210	15,730	14,300
	26 9	1,960	25,480	
	23 6	2,890	37,570	
36	31 4	1,440	18,720	16,500
	28 1	2,250	29,250	
	24 10	3,240	42,120	
38	33 9	1,440	18,720	18,700
	29 5	2,560	33,280	
	26 2	3,610	46,930	
40	35 1	1,690	21,970	20,900
	30 9	2,890	37,570	
	27 6	4,000	52,000	
42	36 5	1,960	25,480	23,700
	32 1	3,240	42,120	
	28 10	4,410	57,330	
44	37 9	2,250	29,250	26,400
	34 6	3,240	42,120	
	30 2	4,840	62,920	
46	40 2	2,250	29,250	29,700
	35 10	3,610	46,930	
	31 6	5,290	68,770	
48	41 6	2,560	33,280	33,000
	37 2	4,000	52,000	
	32 10	5,760	74,880	
50	42 10	2,890	37,570	36,900
	38 6	4,410	57,330	
	34 2	6,250	81,250	
52	45 3	2,890	37,570	40,700
	39 10	4,840	62,920	
	35 6	6,760	87,880	
54	46 7	3,240	42,120	44,600
	41 2	5,290	68,770	
	36 10	7,290	94,770	
56	47 11	3,610	46,930	49,000
	43 7	5,290	68,770	
	38 2	7,840	101,920	
58	50 4	3,610	46,930	53,400
	44 11	5,760	74,880	
	39 6	8,410	109,330	
60	51 8	4,000	52,000	58,300
	46 3	6,250	81,250	
	40 10	9,000	117,000	

* Length = "laying length" for bell and spigot pipe; and face to face of end flanges for flanged pipe. In selecting a meter tube, adopt one whose maximum capacity is 50 to 100 per cent above estimated average flow in the pipe line. If one tube does not have enough range in capacity, place two tubes in parallel in connection with one instrument. Ends may be flange, bell or spigot. Unless otherwise specified flange ends will be made "Master Fitters Standard" for working pressures up to 125 lbs per sq in. and "Manufacturers Standard" for pressures from 125 to 250 lbs To secure prompt delivery, adopt one of these standards. Builders Iron Fdy, Providence, R. I.

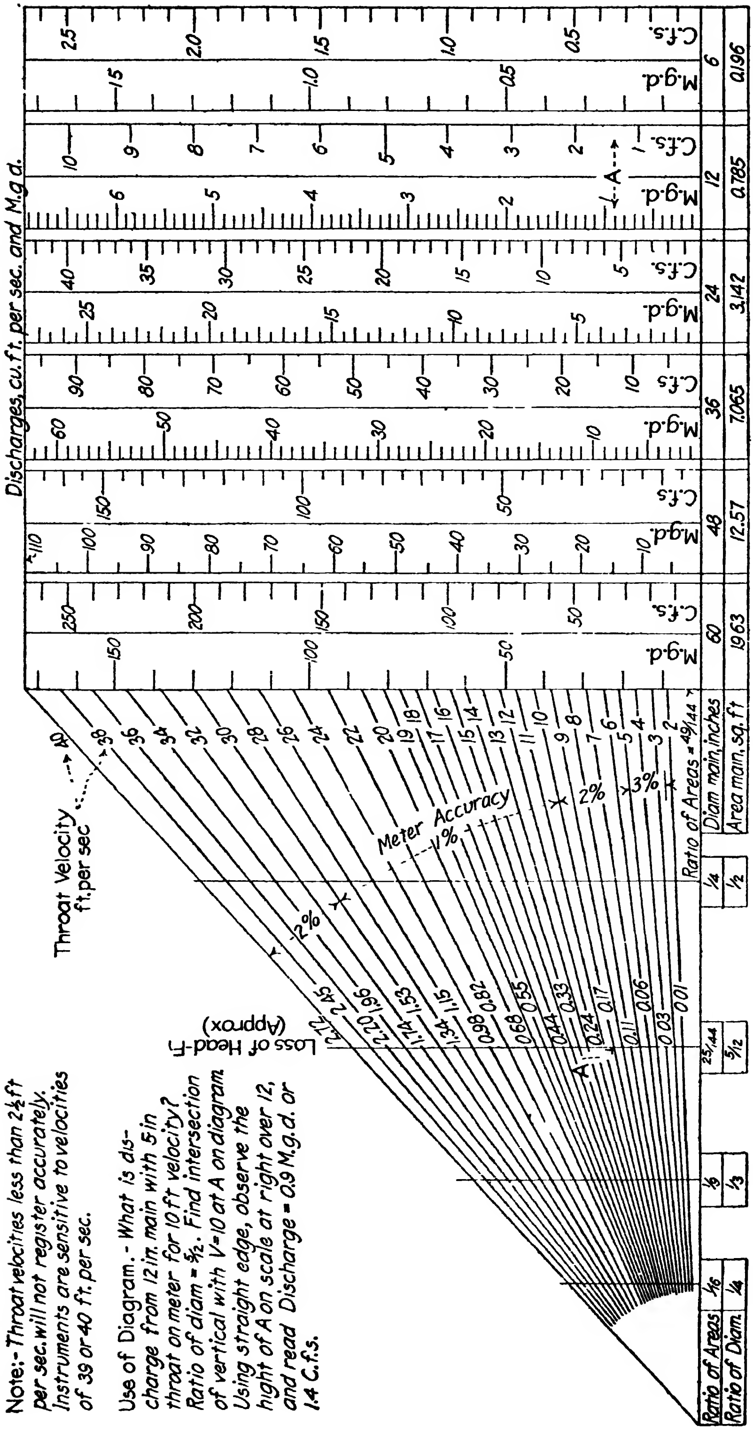


FIG. 265.—Discharge diagram for Venturi meters.

Special Venturi Meter, Catskill Aqueduct delivery system, New York City, was built in a pressure tunnel, concrete lined, 15 ft. diam., carrying 500 mgd. (The tunnel is in rock, 220 ft. below ground at the meter location; the meter is just upstream from a shaft containing a 48-in. riser pipe connecting with the main in the street 10 ft. from downstream end of meter to center line of 48-in. riser.) The minimum loss of head consistent with good measurement was sought. Throat diam. is 104 in., giving ratio of areas of 1 3; throat velocity is 18.4 ft. per sec , with a flow of 700 mgd.; velocity head, 5.2 ft., and loss of head, 0.76 ft. The minimum capacity at which registration can be made reliable with reasonable expense is $(104)^2 \times 10{,}000$ gals. = 108 mgd ; maximum flow is 13×108 or 1404 mgd. The length of the upstream cone = $2\frac{3}{4} \times (180 - 104) = 209$ in. The length of the downstream cone = $11\frac{1}{4} \times (180 - 104) = 855$ in. The length of throat is 104 in. The distance of the center upstream piezometer ring from the end of the upstream cone = $\frac{1}{4} \times 180 = 45$ in. Each piezometer chamber is double, with a separate pipe from each, so that the records of the two can be compared at any time and an idea obtained as to whether a stoppage existed. A handhole is placed at the bottom of each piezometer chamber. Piezometer tubes are 3 in. diam. to permit blowing out in the reverse direction. Steam or carbonic acid gas, such as can be obtained in tanks under pressure, would do this blowing out very effectively, requiring smaller pipes. Piezometer tubes are taken out at midhight and run up on as steep slope as can be obtained to the shaft—nearly 10 per cent (2 or 3 per cent. is generally regarded as sufficient).

Venturi Meter for Reversible Flow. At the top of one shaft of the Catskill aqueduct delivery system, New York, in the connection to a large distributing reservoir, a large capacity Venturi meter capable of measuring flow in either direction is provided The use of this connection will be infrequent. Reversible Venturis of various sizes have been successfully used in other places.

Simplex Water Meter consists of a Venturi tube and two mercury columns, forming a "U." From top of one of these a pipe extends to the full section of Venturi tube, and from top of other a pipe extends to the contracted section.

Table 130. Simplex Meter Dimensions (Venturi Principle)

Size of pipe, in	Capacity in thousand gals per 24 hrs	Approximate shipping weights of Venturi tube, lbs	Lengths over all of Standard Simplex Type Tubes					
			Two flanges,*		Two bells		Bell and spigot	
			ft	in	ft	in	ft	in
4	-----	-----	5	$0\frac{1}{16}$	5	$8\frac{1}{16}$	5	$4\frac{1}{16}$
6	760	315	6	$7\frac{5}{8}$	7	$3\frac{5}{8}$	6	$11\frac{5}{8}$
8	1,360	475	8	$3\frac{1}{8}$	8	$11\frac{1}{8}$	8	$7\frac{1}{8}$
10	2,000	720	9	$10\frac{11}{16}$	10	$7\frac{11}{16}$	10	$3\frac{3}{16}$
12	3,070	1,050	11	$6\frac{3}{16}$	12	$3\frac{3}{16}$	11	$10\frac{11}{16}$
14	4,100	1,475	–	–	–	–	–	–
16	5,400	2,000	14	$9\frac{1}{4}$	15	$7\frac{1}{4}$	15	$2\frac{1}{4}$
18	6,900	2,550	–	–	–	–	–	–
20	8,500	3,260	18	$0\frac{5}{16}$	18	$10\frac{5}{16}$	18	$5\frac{5}{16}$
24	12,300	5,160	21	$3\frac{5}{16}$	22	$1\frac{5}{16}$	21	$8\frac{5}{16}$
30	19,100	8,730	26	$1\frac{15}{16}$	26	$11\frac{15}{16}$	26	$6\frac{15}{16}$
36	27,500	14,550	31	$0\frac{9}{16}$	31	$10\frac{9}{16}$	31	$5\frac{9}{16}$

*Length with two spigots is 6 in greater than for two flanges Venturi tubes are generally made with bronze-lined throats. Approximate shipping weight of register, boxed, is 612 lbs.

Meter registers total number of gallons passed to time of reading. A dial graduated in gallons enables the observer to note rate of flow at moment of observation. A chart recording attachment registers by a perforated line on a ribbon of graduated paper.

TURBINE, DISK AND OTHER METERS

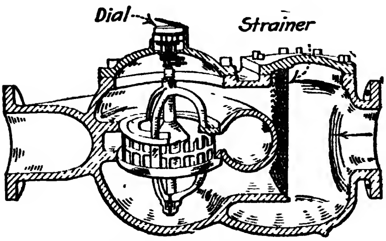


FIG. 266.—Worthington turbine meter.

Worthington Turbine Meters are of current or velocity type, and handle large volumes of water with minimum loss of head. Strainer and moving parts may be gotten at through top covers without removing meter or breaking pipe connections. Heavy pattern turbine meter is adapted for boiler-feed lines measuring hot water under heavy pressure.

Table 131. Sizes and Capacities of Worthington Turbine Meters

Size, in.	Capacity, gal per min		Over-all dimensions, in			Hot Water Turbine Meters				
						Size, in	Horsepower of boilers	Over-all dimensions, in.		
	Normal	Maximum	Length	Width	Hight			Length	Width	Hight
2	150	250	$16\frac{3}{4}$	9	$12\frac{1}{2}$					
3	350	550	24	12	13	3	400–1200	21	$13\frac{1}{2}$	$19\frac{3}{4}$
4	700	1000	27	16	16	4	1200–2500	$24\frac{1}{2}$	$18\frac{1}{2}$	$22\frac{1}{4}$
5	1500	2200	36	22	20					
8	3000	4000	48	29	20	6	2500–5500	32	26	$26\frac{5}{8}$
10	4500	6000	60	35	29					
12	6500	9000	70	42	31					

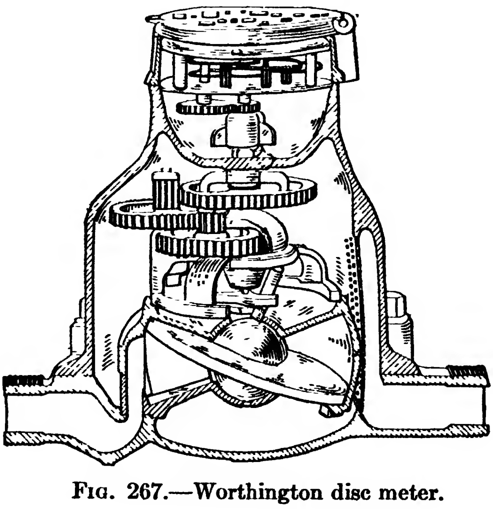


FIG. 267.—Worthington disc meter.

Worthington Disk Meter. Disk is flat, molded under pressure of 2000 lbs. per sq. in. and is water-balanced, eliminating friction. Silt and sand cannot collect under the bearing, as there is an opening under the center of the ball communicating with a settling basin in bottom.

Standard Sapphire Meters (Standard Water Meter Co., Brooklyn, N. Y.). In sizes $\frac{5}{8}$, $\frac{3}{4}$, 1, $1\frac{1}{2}$, 2, 3, 4, 6, 8, 10 and 12 in. Advantages claimed: Can be used for any service; cannot become clogged nor can registration be retarded from sand or sediment; can be accurately repaired without disconnecting from pipe; hot water or excessive heat will not affect them, as no rubber is used; check valves or screens are not required; gear trains are proof against alkali and other injurious ingredients; body of frost-proof meter is fastened together with cast-iron clamps, which are easily adjusted and break at frost pressure of about 200 lbs., thus protecting working parts.

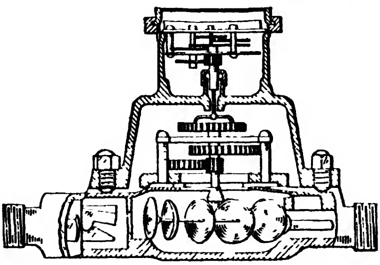


Fig. 268.—Standard Sapphire meter.

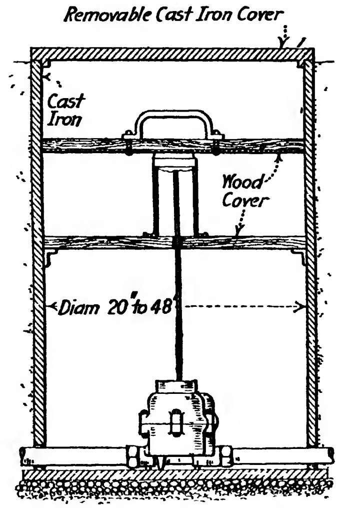


Fig. 269.—Meter box (frost-proof). Obtainable from makers of cast-iron pipe

Bucket Meter. A simple orifice bucket for measuring water, designed by M. L. Enger, is about 42 in. high, 16 in. diam., of galvanized iron, bottom perforated with fifty-six 1-in. holes, with two connections to a glass gage on the side. For this particular bucket discharge in gals. per min. equals 12.8 $N\sqrt{h}$, where N = number of holes open and h = head in ft. Volumes up to 1000 gals. per min. measured fairly accurately—probably within 5 per cent.; portable, cheap and especially adapted for measuring flow of wells. (Ill. W. S. Assn., 1911.) Any similar bucket can be readily calibrated and formula determined by a few simple measurements of its discharge by any convenient means.

"Lea" Water Flow Recorder measures and records graphically flow of water over V-notches and weirs. It is actuated by a float in a chamber directly beneath the instrument; is applicable to measurement of boiler-feed water, discharge from condensers, pump discharges, streams, irrigation ditches, canals, acids, sewage and other fluids, but has an inherent and unavoidable loss of head. Made by Yarnall-Waring Co., Philadelphia. Advantages are: Continuous measurement; no mechanism in contact with water; same instrument used for measuring large or small quantities by using notch plates of various angles, or two or more 90° notches. Guaranteed by makers to be within $1\frac{1}{2}$

per cent. of absolute accuracy by weight; also that average error due to variations in temperature over range of 50° F. will not exceed 0.5 per cent.

Meter Accuracy. Otto Poetsch tested ¾-in. meters, Milwaukee, 1911, none of which had been repaired within 5 yrs.; 3431 meters of piston type gave average slip of 3.15 per cent.; 1955 disk meters gave slip of 0.6 per cent. Piston meters were 15 to 23 yrs. old; disk meters 10 yrs. On basis of these tests, average slip for city was placed at 1 per cent.—(E. R., Jan. 13, 1912.)

In tests at East Orange, N. J., July 29, 1908, to June 22, 1909, two Trident and two Empire meters, out of 18 meters, representing six makes, ran constantly under full pressure for 10½ months, without failure and without serious drop in accuracy. Meters were ⅝-in. size. Tridents passed 1,857,000 and 1,730,000 cu. ft. respectively, equivalent to 13,927,500 and 12,975,000 gals.; Empires 1,289,000 and 1,273,000 cu. ft., equivalent to 9,667,500 and 9,547,500 gals. Differences in quantities were due to varying capacities of the different machines, accuracy being high in all four. When it is considered that the smallest of above quantities is equivalent to the usual supply of a family of 10 for about 48 yrs., or a family of 5 for 95 yrs., the capacity and accuracy of these meters may be appreciated.*

Tests in Des Moines, Iowa, by J. A. Cole, on 1064 meters in service from 1 to 15 yrs., showed an average loss of registration of only 1½ per cent. —(Technology Quarterly, June, 1907.)

Table 132. Summary of Water Metering Statistics in U. S.
(Pittsburg Meter Co.)

Cities: Population From	Cities: Population To	Per Cent taps metered: Under 25%: No. of cities	Under 25%: Consumption per capita gals	Over 75%: No. of cities	Over 75%: Consumption per capita gals
Less than	5,000	24	118	35	33
5,000	10,000	25	110	25	33
10,000	25,000	31	154	20	58
25,000	50,000	16	134	15	57
50,000	100,000	12	151	7	58
Over	100,000	17	182	9	71
		125		111	

Based on returns from 300 cities.—(E. N., Jul. 7, 1910.) See also page 111

Note. The meters mentioned and illustrated above are used merely as convenient examples; there are many makes on the market and much difference of opinion as to their relative merits. Meters should be selected with intelligent regard for the service and conditions for which they are to be used.

* 1909 report, E. Orange Bd. of Water Comm. The Trident is a disk meter, and the Empire of the oscillating piston type.

CHAPTER XXII

PUMPS, PUMPING STATIONS AND EQUIPMENT

Size of Units.* No waterworks pump is operated at full normal capacity all the time. This is important, as efficiency at half-load is much less than at rated capacity, hence the theoretical engine duty obtained on test is not a true measure of results which will be obtained in actual operation. Moreover, as coal used bears some proportion to hours of pumping, coal saved by higher efficiency is necessarily reduced with shorter hours, reduction in operating cost is lessened and there is smaller sum available to offset increased interest charges usual with high duty machines. Stand-by losses reduce net efficiency materially, and, generally, it hardly pays to go into the most expensive forms of pumping machinery when the engine is to be run at a fraction of its rated capacity, or is to be operated only a portion of each day. A common error in design of pumping stations is to subdivide the maximum total pumping capacity required, into pumping units of equal capacity. It will usually develop that a subdivision into different sizes will allow each unit to be operated at full capacity, and far more economical operation will result than would otherwise be possible. By intelligent subdivision, the station may be worked at maximum efficiency for a large proportion of the day.

RECIPROCATING PUMPING ENGINES*

Design of Units.† First determine the theoretical horsepower required to pump the quantity of water desired, against the pressure which it is proposed to maintain. Total head equals static head, plus friction head, plus suction lift. This multiplied by weight of water to be pumped in 1 min. and divided by 33,000 gives theoretical horsepower, or "water horsepower." Then determine mechanical efficiency of the pumping engine from Table 133. Dividing water horsepower by mechanical efficiency gives theoretical indicated horsepower. Then from Table 136, having determined the type of engine to be installed, and knowing its duty, get the number of pounds of steam per i.h.p. required to be furnished by the boilers. It is usually safe to provide an excess for steam consumption of auxiliaries and losses due to condensation in steam pipes, of about 10 to 15 per cent. of the steam required for main pumping units

* Reader is referred to "Direct-Acting Steam Pumps," by F F Nickel, McGraw-Hill Book Co, Inc 1915

† N S Hill, Jr, Proc Mun Engrs of N Y, 1911

Table 133. Mechanical Efficiency of Pumping Engines; Indicated Horsepower in Steam Cylinders*

Capacity U S gallons per 24 hrs.	Total water load against plunger in pounds pressure per sq in. including suction												Mech. eff in per cent. of indicated power
	40	50	60	70	80	90	100	110	120	130	140	150	
	Indicated horsepower of steam cylinders												
1,000,000	20	25	30	35	40	45	50	55	60	65	70	75	80
1,500,000	30	37	44	52	58	66	74	81	90	96	104	111	81
2,000,000	40	50	60	70	80	90	100	110	120	130	140	150	82
2,500,000	48	60	72	82	96	110	120	133	145	160	170	180	83
3,000,000	55	71	84	100	114	128	143	157	171	185	200	214	84
4,000,000	75	94	113	132	150	170	188	207	226	245	264	283	85
5,000,000	93	116	140	161	186	210	234	255	280	304	325	350	86
6,000,000	110	138	166	205	220	248	276	303	330	360	386	415	87
7,000,000	129	161	194	225	257	290	322	355	387	418	450	484	87
8,000,000	146	182	218	255	291	328	364	400	435	474	510	547	88
9,000,000	164	205	245	286	328	368	410	450	492	532	572	614	88
10,000,000	181	225	270	315	360	405	450	495	540	585	630	675	89
11,000,000	198	247	296	346	395	446	495	543	592	642	690	740	89
12,000,000	214	266	320	375	426	480	534	585	640	692	750	800	90
13,000,000	231	289	346	404	464	520	579	638	694	745	809	869	90
14,000,000	246	308	370	430	494	554	617	676	740	800	860	923	91
15,000,000	264	330	396	461	529	593	660	726	790	860	922	990	91
16,000,000	278	348	418	488	556	627	697	767	832	906	975	1,043	92
17,000,000	296	370	444	520	591	666	740	810	888	960	1,034	1,109	92
18,000,000	310	388	465	541	619	696	775	852	930	1,006	1,042	1,162	93
20,000,000	345	430	518	602	689	775	861	946	1,033	1,120	1,205	1,290	93
22,000,000	375	458	560	655	750	843	990	1,030	1,123	1,266	1,310	1,406	94
25,000,000	426	532	639	745	850	959	1,065	1,173	1,278	1,381	1,491	1,596	94
30,000,000	505	631	800	885	1,010	1,138	1,262	1,390	1,519	1,640	1,772	1,895	95
35,000,000	590	738	885	1,033	1,180	1,325	1,478	1,622	1,772	1,918	2,063	2,219	95
40,000,000	668	833	1,000	1,168	1,335	1,500	1,669	1,858	2,025	2,165	2,340	2,500	96

Table 134. Trial Duty Performance of Steam-driven Condensing Pumping Engines*

Type	Duty, million ft -lbs per 1000 lbs dry steam
Vertical, triple expansion, crank and fly-wheel...	140 to 180
Horizontal, cross compound, crank and fly-wheel..	110 to 140
Horizontal, duplex, direct-acting, triple	75 to 100
Horizontal, duplex, direct-acting, compound......	50 to 70
Turbine-driven, centrifugal pumps..............	70 to 140
Engine-driven, centrifugal pumps	70 to 110

Many place too much value upon trial engine duty; it is important, but effectiveness of remainder of plant is of equal consequence. Sight must not be lost of fact that cost of coal, which is the item chiefly affected by high engine duties, is, in most cases, but 30 to 45 per cent. of total operating cost and a much lower per cent of entire cost of pumping.

Proposals for Pumping Engines. Wide open proposals for engines generally result in large variations in the bids. Exempting the engine contractor from all foundation work, etc., generally results in better work and prices. The data to be furnished bidders consist of (*a*) Static water pressure: (1) at engine-room floor; (2) from floor to the level of water in the pump well. (*b*) Allowance for friction: (1) force main; (2) suction pipe. (*c*) Total working load upon plungers. (*d*) Steam pressure at the throttle. (*e*) Available clear hight above the engine-room floor. (*f*) Vertical distance from engine-

* N. S Hill, Jr, Proc. Mun. Engrs. of N. Y., 1911.

room floor to basement floor. (g) Wall to wall of engine room, inside, both directions. (h) Available space on floor across the engine room. (i) Available on floor lengthwise of engine room. (j) Available in basement, lengthwise of engine room. (k) Distance from building wall to pump well. Furnish bidders plans and sections of the building, especially if space is cramped or obstructed. (l) Air chamber required at the inboard end of the suction pipe. (m) Air chamber required for the force main. (n) Capacity. (o) Number of units. (p) Transportation and erection facilities, sizes of entrances. (q) Special local conditions if any.—(C. A. Hague, Proc. Am. W. W. Assn., 1905, p. 150.)

On Cost of Pumping Engines* no really reliable figures can be given. Such figures vary with type, size, head pumped against, steam pressure, supply and demand, and cost of materials. If in doubt as to type needed, get bids on types to be considered. To fix upon cheapest unit for a particular case consider costs of the following: (a) engine, (b) foundations, (c) land, (d) buildings, (e) boilers necessary to supply steam to engine, (f) coal per ton, (g) maintenance and repairs, (h) lubricants and waste, and (i) any other elements affected by type of engine.

Cost of coal can be determined from following formulæ: Where guaranteed duty of engine is known, and expressed in foot-pounds per 1000 lbs. of steam, pounds of steam per hr. per i.hp. $= 60 \div \frac{\text{duty}}{33{,}000 \times 1{,}000} = \frac{1{,}980{,}000{,}000}{\text{duty}}$.

Steam consumption corresponding to various duties may also be obtained from Table 136, page 464, which was computed from above formula.

$$\text{Annual cost of fuel} = \frac{S \times \text{i.hp.} \times T \times C}{E \times 2000}.$$

S = lbs. of steam used by engine per hp. per hr.;
i.hp. = average indicated horsepower developed;
T = number of hours during year pumping is required;
C = cost of coal in dollars per ton of 2000 lbs.;
E = evaporation factor of boiler, approximately 8 lbs. of water evaporated per lb. of ordinary good bituminous coal.

To cost of coal thus determined should be added 4 to 6 per cent. interest, on cost of engine, 8 to 10 per cent. for repairs, depreciation, etc., and 1 to 2 per cent. refunding sinking fund.* For small quantities of water, fuel item usually becomes a secondary consideration, in selecting types of pump. Even when coal is $1350 per year for each million gals. pumped per day, total coal bills do not amount to much for 2,000,000 or 3,000,000 gals. per day; but when quantity gets toward 10,000,000 gals., they are more important. The larger triple-expansion pumping engines of reciprocating displacement type pump 1,000,000 gals. per day with $625 worth of coal per year for best records, and $900 is an ordinary good record; while $1350 is about the best steam-turbine pumping record†.

* N. S Hill, Jr, Proc. Municipal Engrs of N Y, 1911.
† Chas Arthur Hague, Trans A. S. C. E, Vol. 74, 1911.

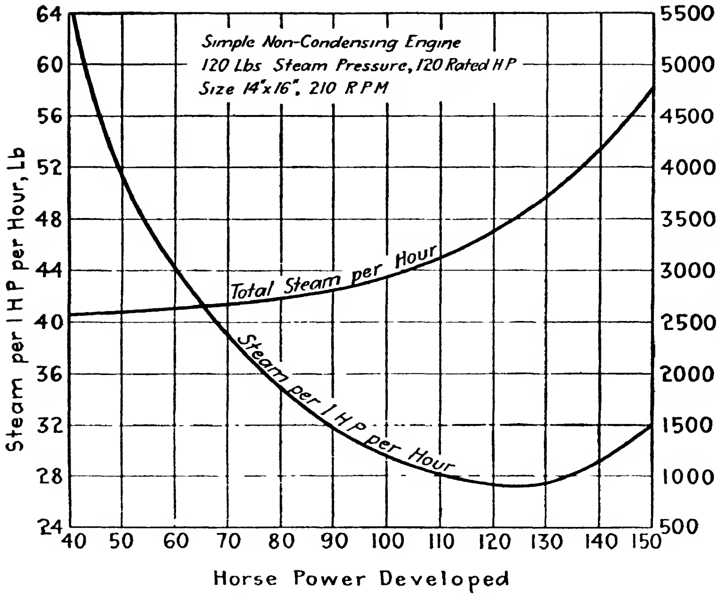


FIG. 270 —Steam consumption under variable load
(N S Hill, Jr, Proc Municipal Engrs, N Y, 1911)

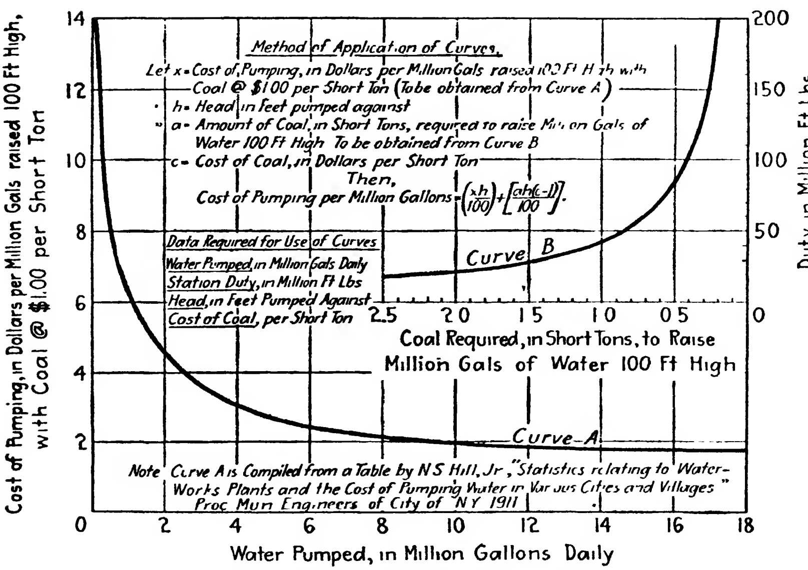


FIG. 271.—Cost of pumping. (See Table 135.)

Table 135. Pumping Statistics and Cost of Pumping at Waterworks Plants in Various Cities and Villages. N. S. Hill, Jr.

No	Pumping statistics			Cost		No	Pumping statistics			Cost	
				Based on pumping station expenses						Based on pumping station expenses	
	Pumpage for the year, million gals	Aver head pumped against, ft	Duty per 100 lbs of coal, million ft -lbs	Per million gals pumped	Per million gals raised 1 ft high		Pumpage for the year, million gals	Aver head pumped against, ft	Duty per 100 lbs of coal, million ft -lbs	Per million gals pumped	Per million gals. raised 1 ft high
1	32 8	185	8 10	$56 85	$0 31	27	855 2†	263	90 89	11 84	0 04
2	39 7*	322	29 66	57 31	0 18	28	856 3*	302	98 00	11 57	0 04
3	53 3†	289	31 92	30 11	0 10	29	961 0	172	44 40	15 75	0 09
4	67 9†	240	23 43	41 05	0 18	30	1,088 0	401	53 10	20 65	0 05
5	70 5	389	22 55	44 36	0 11	31	1,095 0	210	67 70	15 27	0 07
6	73 0	115	6 90	83 02	0 72	32	1,104 7†	204	86 65	12 63	0 06
7	95 0	321	88 01	24 78	0 08	33	1,155 6	217	49 40	15 89	0 07
8	105 8	230	46 90	11 15	0 04	34	1,206 4	186	76 48	10 80	0 06
9	118 2	280	38 22	30 29	0 11	35	1,322 0	120	64 27	5 68	0 05
10	158 4*	252	40 92	29 10	0 12	36	1,404 6†	128	103 22	4 65	0 04
11	205 5†	205	38 18	20 79	0 10	37	1,561 4‡	240	54 79	8 35	0 04
12	208 9	176	41 37	18 50	0 10	38	1,909 8	164	78 34	16 29	0 10
13	237 2	322	46 45	25 75	0 08	39	2,693 5†	130	123 76	5 22	0 04
14	246 1†	232	32 85	26 53	0 11	40	2,718 3†	184	134 76	6 61	0 04
15	250 0	70	25 56	11 51	0 16	41	2,905 4*	287	95 88	5 28	0 02
16	262 6	98	30 80	23 92	0 21	42	2,920 0	180	29 80	21 55	0 12
17	272 8*	182	43 51	17 12	0 09	43	3,659 5*	235	72 33	4 51	0 02
18	350 0	179	17 90	18 06	0 10	44	3,678 6	189	125 08	4 69	0 02
19	364 9*	316	34 51	24 74	0 08	45	4,188 1	437	104 91	11 84	0 03
20	372 3	199	31 60	25 08	0 13	46	7,957 9	112	60 50	2 82	0 02
21	438 0	185	42 90	21 78	0 12	47	9,111 8	199	111 77	3 44	0 02
22	502 2*	140	62 53	13 67	0 10	48	10,708 5†	130	138 84	3 03	0 02
23	542 6*	246	97 94	10 58	0 04	49	11,293 9	164	102 46	4 44	0 03
24	580 1	292	76 68	19 25	0 07	50	17,130 3	126	116 18	3 10	0 02
25	617 3	251	35 23	12 69	0 18	51	19,183 4†	46	105 38	1 68	0 04
26	660 4†	216	48 29	15 22	0 07	52	27,816 8†	132	87 95	3 33	0 01

* Without slip allowance † With slip allowance ‡ Imperial gallons see page 623

Cost Complete, with Foundations, Piping and Appurtenances, per 1,000,000 *Gals.*: 24-*hr. Capacity.** Compound-condensing, low-duty, horizontal, $2300 Low-duty, triple, condensing, horizontal, $2800. Cross-compound, condensing, horizontal, $3300. High-duty triple, condensing, vertical, $4800. First and second are non-rotative, or direct-acting; third and fourth are crank-and-fly-wheel, figures do not include buildings, land, chimneys, wells, boilers, etc. Boilers with mechanical stokers, feed-pumps, appurtenances and steam piping, ready for service under average conditions, would be covered by $20 per boiler horsepower. These averages are based on: Total water load against plungers, 90 lbs. per sq. in., or 207 ft. head, including suction and friction; actual evaporation in boilers under working conditions, 8 lbs. of water per lb. of coal, with feed at 150° F., coal at $3 per ton of 2000 lbs. Steam pressure at engine throttle, 75 lbs. gage, for low-duty compound; 125 lbs. gage, for low-duty triple and cross-compound; 150 lbs. gage, for high-duty triple; allowance of 5 lbs. above pressures given for boiler pressures.

*Chas Arthur Hague, Trans A S C E, Vol 74, 1911

Table 136. Boiler Horsepower Required for each Pump Horsepower, Counting 10 sq. ft. of Heating Surface per Boiler Horsepower*

Duty in ft -lbs per 1000 lbs oi dry steam	Boiler hp per pump hp.	Lbs of steam per hr. per pump hp	Duty in ft.-lbs per 1000 lbs. of dry steam	Boiler hp. per pump hp.	Lbs. of steam per hr. per pump hp.	Duty in ft -lbs per 1000 lbs of dry steam	Boiler hp. per pump hp	Lbs of steam per hr. per pump hp
40,000,000	1 63	49 5	120,000,000	0.55	16.5	165,000,000	0 40	12 0
50,000,000	1.32	39.6	125,000,000	0.52	15.8	170,000,000	0.39	11.6
60,000,000	1 10	33 0	130,000,000	0 51	15.2	175,000,000	0 38	11 3
70,000,000	0.94	28 4	135,000,000	0.49	14.7	180,000,000	0 37	11 0
80,000,000	0.83	24 7	140,000,000	0.47	14.1	185,000,000	0 36	10 7
90,000,000	0.74	22 0	145,000,000	0 46	13 6	190,000,000	0 35	10 4
100,000,000	0 66	19 8	150,000,000	0 44	13 2	195,000,000	0 34	10 0
110,000,000	0 60	18 0	155,000,000	0 43	12.8	200,000,000	0.33	9 9
115,000,000	0 57	17 2	160,000,000	0 41	12 4	....	...	

Table 136 is based on 1 sq. ft. of heating surface evaporating 3 lbs. of water per hr., from 150 ° (temperature of feed), into steam at 150 lbs. gage pressure. This is safe in most cases, but any desired increase may be made. For example: If 2½ lbs. water per sq. ft. heating surface per hour are all it would be safe to reckon on, then 20 per cent. added to boiler hp. of Table 136 would provide for such a case.*

Table 137. Cost of Complete Pumping Stations* (Reciprocating Engines)

Pressure of water load pumped against. lbs per sq in	Cost of plant, per million gals capacity, including reserve	Pressure of water load pumped against· lbs per sq in	Cost of plant, per million gals. capacity, including reserve
30	$6,750	90	$8,250
40	7,000	100	8,500
50	7,250	110	8,750
60	7,500	120	9,000
70	7,750	130	10,000
80	8,000	. .	...

Figures given above are closely approximate, so close that it would be taking chances to guarantee results without investigating each case. The work contemplates best type of modern, triple-expansion pumping engines, and high-pressure boilers. Buildings are assumed of good design and quality; of brick or of stone where stone is cheap; roofs steel-trussed and slate-covered; chimneys adequate; and intakes properly proportioned and thoroughly screened. Cost includes everything except land.

Items for and against High-duty Pumping Plant*

Against high-duty:	In favor of high-duty.
Maintenance account for machinery	Maintenance account for boilers
Interest on machinery	Interest on boilers
Oil, waste, packing, etc.	Sinking fund for boilers
Sinking fund for machinery	Coal account

*C. A. Hague, T. A. S C. E., Vol. 74, 1911, and N. S Hill, Jr.

Table 138. Fixed Charges against Pumping Machinery and Boilers

Vertical triple-expansion pumping engines		All other types of reciprocating pumping engines	
Maintenance account...	2%	Maintenance account............	3%
Interest account...	4%	Interest account....	4%
Sinking fund account..	3%	Sinking fund account.	5%
Oil, waste, packing, and small repairs	1%	Oil, waste, packing, and small repairs..	1%
Total fixed charges	10%	Total fixed charges.	13%

For vertical triple pumping engines life is taken at 33⅓ yrs., and for all other types at 20 yrs.

Table 139. Fixed Charges against Boiler Plant*

Maintenance account....	5%
Interest account...	4%
Sinking fund account	5%
Total fixed charges	14%

Comparative Economy of High-class Pumping Machinery and Fuel.* Two steam pumping plants recorded 1.02 lbs. coal per indicated-horsepower-hour, and 1.98 lbs. Both records were obtained as nearly as could be under similar conditions in actual waterworks pumping; latter fuel was slack at $1.50 per net ton; and former anthracite coal at $4.50. Heat units developed by slack, on analysis, 11,000 per lb.; heat units developed by coal, 14,000 per lb. In the slack the buyer obtained 146,000 heat units for 1 ct.; in the anthracite 62,000. The plant consuming slack used 363 heat units per horsepower-minute, while the plant with anthracite consumed 238. Efficiency of boilers was 70 per cent. with slack and 80 per cent. with anthracite. This, reduced to work demonstrated by indicated-horsepower, gives 13,200,000 ft.-lbs. for 1 ct. for anthracite and 9,806,000 ft.-lbs. for slack, with efficiencies of boilers equalized, and shows a difference of 3,394,000 ft.-lbs. in steam economy of engines. The engine with slack gave 130,000,000 ft.-lbs. per million B.t.u., and that with coal gave 163,000,000, further demonstrating the differ-

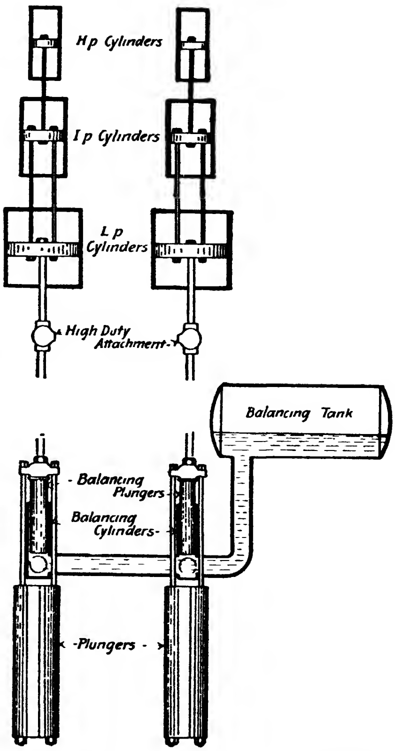


FIG. 272.—Working parts of Worthington vertical triple-expansion high-duty engine.

*C. A. Hague, T. A. S. C. E., Vol. 74, 1911.

ence in steam economy in the two machines. Gain in foot-pounds for 1 ct. with engine using anthracite over other is 34 per cent., and gain on heat-unit basis is 26 per cent., which indicates good judgment in putting enough investment into engine, boilers ($4166 per million gals. capacity per 24 hrs.), and fuel to obtain good results; the engine working on slack cost $4083; prices do not include foundations and appurtenances. Present high-type waterworks pumping engine apparently cannot be improved to any material extent in principle; best to be expected is extremely small increase in economy by improved construction. It is probable an increase in fuel economy from 6 to 12 per cent. can be gained in many steam plants by use of superheated steam. If there is waste heat in smoke-flues or uptakes of boilers, reheaters for receiver steam can be provided, and this steam made a vehicle for transportation of heat now getting away up the chimney back to the engine, and there made to do work.

Economy of Small Steam Engines.* Steam engines in economy vary from 12 to 60 lbs. of feed water and from $1\frac{1}{2}$ to 7 lbs. of coal per hr. per indicated horsepower. Locomotives average 3000 gals. of water per 100-mi. run.

Table 140. Economy of Steam Engines*

Type	Feed water temperature, Fahr.	Pounds of water evaporated per lb. of coal	Pounds of steam per 1 hp. used per hr.	Pounds of coal used per 1 hp. per hr.	Cost per 1 hp. per hr.
Non-condensing	210°	10.5	29.0	2.75	$0.0073
Condensing	100°	9.4	20.0	2.12	0.0056
Compound jacketed	100°	9.4	17.0	1.81	0.0045
Triple-expansion jacketed	100°	9.4	13.6	1.44	0.0036

Cumberland coal used; cost assumed $6 per ton. See also p. 161.

Best Reciprocating Pumping Plants† involve: Vertical, triple-expansion, crank-and-fly-wheel pumping engines, long stroke, rotative speed not to exceed 20 r.p.m.; maximum piston travel 200 ft. per min., modified steam-jacketing and reheating; steam pressure at throttle, 175 lbs. gage; moderately superheated steam by independent apparatus; smoke-flue reheating; water-tube boilers, mechanical stokers; natural draft at least 0.8 in. of water; feed-water economizers; automatic damper regulators, coal bought on basis of 14,000 heat units per lb.; boiler efficiency of 75 per cent.; coal per indicated-horsepower, 1 lb. for large plants, 1.75 lbs. for small plants, maintenance of engines, 1.5 per cent. for large plants, 3 per cent. for small plants. Quadruple-expansion pumping engines, resulting either in a tandem arrangement, or in abandonment of three-plunger designs so favorable to uniform hydraulic effects, are not suitable for waterworks service. Mechanical efficiency of displacement machinery is 93 per cent.

Benefit of superheat† in connection with engines of the highest type is probably 6 per cent., as the upper limit of fuel economy that can be expected.

* Geo. F. Blake Mfg. Co.
† C. B. Buerger, T. A. S. C. E., Vol. 74, 1911.

Table 141. Duty and Consumption of Fuel of Steam Pumping Plants (Dow)

S = pounds of water per hp.-hour, S_1 = pounds of coal per hp.-hour; E = mechanical efficiency of plant, per cent, $S_1 = (S \times 100) \div E$, find result in second column. For duty and fuel consumption find values opposite. Fuel is based on raising 100 gals per min, 100 ft high for 24 hrs.

Duty in million foot-pounds per 1000 lbs. dry steam	Steam consumption per water-horsepower, lbs per hr	Coal in tons per 24 hrs			Oil in barrels, 42 gals, per 24 hrs.			
		Evaporation*			Evaporation†			
		6 to 1	8 to 1	9 to 1	12 to 1	13 to 1	14 to 1	15 to 1
30.3	65.0	0.328	0.246	0.220	0.93	0.86	0.80	0.74
33.0	60.0	0.302	0.227	0.204	0.86	0.79	0.74	0.685
36.0	55.0	0.280	0.208	0.186	0.79	0.73	0.68	0.63
38.7	51.0	0.258	0.193	0.172	0.73	0.67	0.63	0.58
44.5	44.5	0.224	0.167	0.150	0.63	0.60	0.55	0.50
50.0	39.6	0.199	0.149	0.133	0.57	0.52	0.485	0.45
55.5	35.7	0.179	0.134	0.120	0.51	0.47	0.44	0.40
60.5	32.5	0.164	0.122	0.109	0.46	0.43	0.40	0.37
66.5	29.7	0.149	0.112	0.100	0.42	0.39	0.365	0.34
72.0	27.5	0.138	0.104	0.092	0.39	0.36	0.34	0.315
77.5	25.5	0.128	0.096	0.085	0.36	0.325	0.315	0.29
82.5	23.8	0.120	0.090	0.081	0.34	0.315	0.293	0.272
88.0	22.4	0.112	0.085	0.075	0.32	0.295	0.275	0.255
94.0	21.0	0.105	0.080	0.071	0.30	0.275	0.26	0.24
100.0	19.8	0.100	0.075	0.067	0.29	0.26	0.245	0.225
105.0	18.8	0.095	0.072	0.061	0.27	0.25	0.23	0.215
110.0	18.0	0.091	0.068	0.059	0.25	0.235	0.22	0.205

* Ratio of water evaporated to dry coal consumed
† Ratio of lbs. of water evaporated to lbs. of oil consumed

Safe Station Duty. The following facts are to be considered in arriving at safe station duty: (*a*) Capacity and head. (*b*) Boilers. Two boilers per battery are used in the pumping stations of New York City. (*c*) Method of firing. (*d*) Grade of coal. (*e*) Pump efficiency. (*f*) Carefulness of maintenance and operation. (*g*) Efficiency of attendants and the general standards of the department. Assume 115,000,000 ft.-lbs. duty per 100 lbs. of coal for large stations, for approximate estimates.

Altitude, Pressure and Suction. The hight that a pump will lift water depends on the atmospheric pressure, which decreases as the altitude increases, roughly 0.5 lb. per sq. in. for every 1000 ft. of ascent. At altitude 1500 ft., the pressure is 0.75 lb. less than at sea level, or roughly 14 lbs. per sq. in. At sea level one atmosphere has a pressure of 14.696 lbs. per sq. in., or a head of 33.94 ft. of water, or 29.922 in. of mercury. Moisture in the air makes a difference in the pressure. The readings of a barometer will take care of both altitude and moisture corrections.

Table 142. Pressures Corresponding to Barometer Hights

Barometer, in.	Pressure per sq. in., lbs.	Barometer, in.	Pressure per sq. in., lbs.
28	13.74	29¾	14.60
28¼	13.86	30	14.72
28½	13.98	30¼	14.84
28¾	14.11	30½	14.96
29	14.23	30¾	15.09
29¼	14.35	31	15.21
29½	14.47		

To find the theoretical lift of a pump in ft., multiply the pressure per sq. in. from the barometer reading by 2.3. Of course the pump will not lift this hight, as the pump cannot produce a perfect vacuum, and even if it could

Table 143. Theoretical Horsepower Required to raise Water to Different Hights*

Feet / Gal per min	Mgd.	5	10	15	20	25	30	35	40	45	50
5	0 007	0 006	0.012	0 019	0.025	0.031	0.037	0.044	0 05	0 06	0 06
10	0 014	0 012	0.025	0.037	0.050	0 062	0 075	0 078	0 10	0.11	0 12
15	0 022	0 019	0.037	0.056	0.075	0 094	0 112	0 131	0 15	0 17	0 19
20	0.029	0.025	0.050	0 075	0.100	0.125	0.150	0.175	0 20	0 22	0.25
25	0 036	0.031	0.062	0.093	0.125	0.156	0.187	0.219	0.25	0.28	0.31
30	0.043	0.037	0.075	0.112	0 150	0.187	0.225	0 262	0 30	0 34	0.37
35	0 050	0 043	0.087	0 131	0.175	0 219	0.262	0 306	0 35	0 39	0.44
40	0 058	0 050	0.100	0 150	0 250	0 300	0.300	0.350	0 40	0 45	0 50
45	0.065	0.056	0.112	0 168	0 225	0 281	0 337	0 304	0 45	0 51	0 56
50	0.072	0.062	0.125	0 187	0.250	0.312	0.375	0.437	0.50	0 56	0 62
60	0 086	0.075	0.150	0 225	0 300	0.375	0.450	0.525	0 60	0 67	0 75
75	0 108	0 093	0 187	0 281	0 375	0 469	0.562	0 656	0 75	0 84	0 94
90	0 129	0 112	0 225	0 337	0 450	0 562	0 675	0 787	0 90	1 01	1 12
100	0 144	0 125	0.250	0 375	0 500	0 625	0 750	0 875	1 00	1 12	1 25
125	0 180	0.156	0.312	0 469	0.625	0.781	0.937	1.094	1.25	1 41	1.56
150	0 216	0.187	0 375	0 562	0 750	0 937	1.125	1 312	1.50	1 69	1 87
175	0 252	0 219	0 437	0 656	0 875	1 094	1.312	1 531	1 75	1 97	2 19
200	0 288	0 250	0.500	0 750	1.000	1 250	1 500	1 750	2 00	2 25	2 50
250	0 360	0 312	0 625	0 937	1 250	1.562	1.875	2.187	2 50	2.81	3 12
300	0 432	0 375	0.750	1.125	1.500	1.875	2.250	2.625	3.00	3.37	3 75
350	0 504	0 437	0 875	1.312	1 750	2 187	2 625	3.062	3 50	3.94	4 37
400	0.576	0 500	1 000	1 500	2 00	2 500	3 000	3 500	4 00	4 50	5.00
500	0.720	0.625	1.250	1 875	2.500	2.125	3.750	4 375	5.00	5.62	6 25

Feet / Gal. per min	60	75	90	100	125	150	175	200	250	300	350	400
5	0 07	0 09	0 11	0.12	0 16	0 19	0 22	0 25	0 31	0 37	0.44	0.5
10	0 15	0 19	0.22	0.25	0 31	0 37	0 44	0 50	0 62	0 75	0 87	1.0
15	0 22	0 28	0.34	0 37	0 47	0 56	0.66	0.75	0 94	1.12	1.31	1.5
20	0 30	0 37	0 45	0 50	0 62	0.75	0 87	1.00	1 25	1 50	1 75	2 0
25	0.37	0 47	0.56	0.62	0 78	0 94	1.09	1.25	1.56	1.87	2.19	2 5
30	0 45	0 56	0 67	0 75	0 94	1.12	1.31	1.50	1 87	2.25	2.62	3.0
35	0 52	0 66	0 79	0 87	1 08	1 31	1.53	1.75	2.19	2 62	3.06	3 5
40	0 60	0 75	0 90	1 00	1 25	1 50	1.75	2 00	2 50	3 00	3.50	4 0
45	0 67	0 84	1.01	1.12	1 41	1 69	1.97	2 25	2.81	3.37	3.94	4 5
50	0.75	0.94	1.12	1.25	1.56	1.87	2.19	2 50	3.12	3.75	4.37	5 0
60	0 90	1 12	1 35	1 50	1 87	2.25	2.62	3 00	3.75	4 50	5 25	6 0
75	1 12	1 40	1.69	1 87	2.34	2.81	3 28	3.75	4.69	5 62	6.56	7 5
90	1 35	1 68	2.02	2.25	2 81	33.7	3.94	4.50	5 62	6 75	7 87	9 0
100	1 50	1 87	2 25	2 50	3 12	3 75	4 37	5.00	6 25	7.50	8 75	10 0
125	1 87	2.34	2.81	3.12	3.91	4.69	5.47	6.25	7.81	9.37	10.94	12 5
150	2 25	2 81	3.37	3.75	4 69	5.62	6.56	7.50	9 37	11 25	13.12	15.0
175	2 62	3 28	3.94	4 37	5 47	6 56	7.66	8 75	10.94	13 12	15.31	17 5
200	3 00	3 75	4 50	5 00	6 25	7.50	8.75	10 00	12 50	15 00	17 50	20 0
250	3 75	4 69	5 62	6 25	7 81	9.37	10.94	12 50	15 72	18 75	21.87	25 0
300	4.50	5 62	6.75	7.50	9.37	11.25	13.12	15.00	18.75	22 50	26.25	30.0
350	5 25	6 56	7.87	8.75	10 94	13.12	15.31	17.50	21.87	26 25	30.62	35 0
400	6 00	7.50	9 00	10.00	12.50	15.00	17.50	20.00	25 00	30 00	35.00	40 0
500	7.50	9.37	11.25	12.50	15.62	18.75	21.87	25.00	31 25	37.50	43.75	50 0

The theoretical horsepower required to elevate water is found by multiplying gals. pumped per min. by total lift (including friction) in ft., and dividing by 4000. Plus error of 2.8 per cent.

* Dean Steam Pump Works, Indianapolis.

there must be enough difference between the pressure in the pump chamber and the atmosphere not only to sustain the hight of the column, but to overcome its friction. For tables of pressures, see p. 626.

SMALL RECIPROCATING PUMPS

Selecting Small Pumps. When selecting small pumps answers to following questions are needed: (1) For what purpose is pump to be used? (2) What liquid to be pumped? Hot or cold, clear or gritty, fresh or salt? (3) From what source is supply to be taken? (4) Maximum quantity to be pumped per hour (or day or minute)? (5) To what hight is liquid to be lifted by suction? Length and diam. of suction pipe, and number of turns? (6) To what hight, or against what pressure is liquid to be forced? Length and diam. of delivery pipe, and number of turns? (7) Pressure of steam? (8) Exhaust

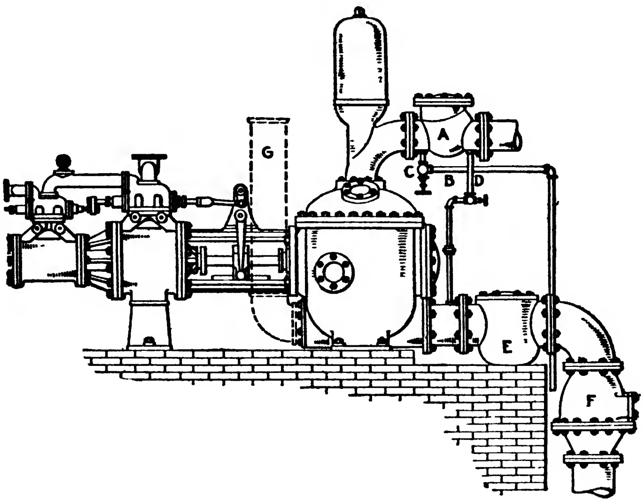


FIG. 273.—Pumping engine. Arrangement of pipe connections. (Dow.)
(Text on p. 470)

into atmosphere, condenser or closed pipe? If latter, what will be back pressure? (9) To be situated in tunnel, well, or on surface? (10) To be driven by direct connection, belt, gearing, or otherwise? (11) Number of revolutions of shaft from which power is to be taken? (12) When electric motors are to be used, state whether current is direct or alternating, and give voltage. If alternating, of how many phases? How many cycles per second? Order a pump large enough; slow piston speed is desirable, especially when pumping against heavy pressure. For hot water, a pump at least half larger than for cold water should be used. At various altitudes, heat at which steam rises should be considered.

Pumps of the piston type, owing to the facility with which the packing can be renewed, and the smaller clearance spaces in the pump cylinders, are particularly efficient for lifting water by suction, where circumstances are such that it is impossible to prime the suction piping before starting the pump.

Pumping Engine Pipe Connections. The check valve *A* (see Fig. 273) keeps water in the delivery pipe when the pump is open for inspection; also relieves the pressure on the delivery valves when starting. It is very common for a pump, notwithstanding the suction pipe being tight, to refuse to lift water by suction when the delivery valves are held down by the pressure of the water in the delivery pipe; air in the pump chamber is thus unable to escape but is merely compressed. Waste pipe *B* frees the pump of air, should it be present in the cylinders; this pipe is led from the end of the check valve next the pump into the sump, or any other convenient place. When starting, valve *C* should be open to allow the air to escape, and when properly started, should be closed. *D*, a by-pass pipe, connecting the delivery pipe, beyond the check valve, to the suction pipe, is for filling the pump cylinders and suction pipe with water from the delivery column should they have been previously emptied. *E* is a strainer box in the suction pipe, handy for inspection. *F* is the suction check valve, easily reached, for retaining water in the pump cylinders and suction pipe so that when starting the pump will not have to free itself of air. This is essential especially when the suction pipe is long or the suction lift high, as is also the suction or vacuum chamber, *G*.

Directions for Installing and Operating Pumps. *Location.* A pump should be located as near the source of supply as circumstances will permit, particularly if the water or other fluid is to be drawn into the pump by suction; as the first requisite is that a full and steady supply must be furnished to allow the pump chamber to fill, in order to effect a steady motion of, and constant delivery from, the pump. This is important as no pump can work satisfactorily that does not have a full supply of water. Place the pump on a masonry foundation in preference to a wooden floor; the latter is liable to rot and bring undue strain on the pipes, causing leaky joints.

Suction Pipe. The suction pipe should in no case be smaller than the size given in the manufacturer's table; if long, it must be larger, as the friction caused by the unusual length will partly overcome the head due to the vacuum, and prevent a full supply from entering the pump; it must be as free as possible from elbows and valves, as they retard flow of water much more than length does. Avoid the possibility of air pockets. The pipe should be air-tight, as a small leak will greatly impair the working of the pump. A foot-valve should be used on long or high suctions, and if used, its net opening should be somewhat in excess of the area of the pipe. If there is danger of foreign substances entering the pipe, a strainer is necessary, which should have an aggregate area of openings, according to the speed at which the pump is to run, from 3 to 6 times the area of the pipe.

Suction Air Chamber. A large air chamber on the suction line close, or connected, to the suction chamber of the pump is always an advantage where the suction is great, as it tends to equalize the flow and to prevent concussion in the pump. It is necessary for high-speed pumps. It should be so located as to relieve pulsation in the suction line in the most efficient manner. If the water cylinder has suction openings on both sides, place the air chamber opposite the suction pipe; but if the cylinder has a suction opening on one side only, the suction pipe may be placed on the same side of the cylinder as the air chamber.

The suction air chamber is made from a piece of pipe of the same size as the suction pipe, and from 2 to 3 ft. long, capped on the top and fastened to the water cylinder by a short nipple and an elbow or T.

Delivery Pipes should not be smaller than given in the manufacturer's tables, unless for temporary use and short distances; if the distance is great, use a size larger; the difference in first cost will be more than compensated for by a reduced friction, and consequent economy of power. Run every pipe in as direct a line as practicable, and where turns are necessary, use full round bends or elbows, of as large radius as convenient. Use Y-branches in preference to T's. Gate, or straight-way, valves should always be used for water.

Steam and Exhaust Pipes should be straight and as free as possible, and as large as the sizes given in the manufacturer's tables. Before connecting a pump (or engine) blow out the steam pipe thoroughly. Any dirt carried into a steam cylinder will injure it seriously. In connecting make due allowance for expansion of steam pipes when heated by steam. A throttle valve should be placed in a steam pipe as close to the pump as possible. Means should be provided for draining this pipe before starting the pump. Placing a small feed-water heater in exhaust pipe from pump, or putting a three-way cock in exhaust piping and connecting the exhaust piping to the suction pipe of the pump is sometimes suggested, but is inadvisable on account of presence of oil in steam. Steam and exhaust pipes should be connected so that they may be drained of the water of condensation.

Graphite pipe joint compound is the best to use wherever there is any possibility of the joint being taken apart at some future time. It contains flake graphite. Due to its lubricating properties, a screwed joint may be made much tighter than with a cement-like preparation. It is equally valuable for threaded or flanged joints of steam, water, gas and air piping, for bolts, nuts, stay-bolts, studs, caps, boiler plugs, hand-hole and man-hole plates of boilers, doors of gas retorts, metal gaskets, flanges, ground joints, etc. It makes the tightest joints, prevents rusting, or corrosion, does not get hard or brittle, but allows the parts to be opened at ease any time. Flake graphite compound is about $2\frac{1}{2}$ times greater in bulk than red or white led, mixed, and is, therefore, pound for pound, much more economical.

Care. If the pump is to be left idle for some time, fill the oil cup, then open the cock from the oil cup so that oil can flow into the steam chest; then let the pump make half a dozen quick strokes to distribute the oil well over the inside of the steam end, and so prevent rusting while the pump is standing. In cold weather open all cocks and drain plugs to prevent freezing when the pump is not in use. Use good cylinder oil only. Keep the stuffing boxes clean, full of good packing, well oiled, and just tight enough to prevent leakage without excessive friction. Don't screw the glands too tightly, and don't allow the same packing to remain in the stuffing boxes too long or it becomes hard and scratches the piston. If the pump runs badly make sure that the water valves and pipes are all right, before examining the steam end. Always see that the pump has a full and steady supply of water (or other fluid). All parts of stock pumps by the best manufacturers are made to standard gages and templates and are interchangeable.

Starting. After pump has made a few strokes exhausting to the atmosphere, three-way cock on exhaust may be turned and exhaust conducted to condenser. In the case of a light suction lift, a piston or plunger pump will readily pick up water in the suction pipe. Before starting a pump when first set up, see that it is not packed too tight; test the valve motion to see that it is free. If the movement of the pump should not be uniform, one stroke with the other, or different parts of a stroke, or refuse to take water from the suction, it will be found that either some substance has lodged under the valves to prevent their closing, or that the pump gets air through some leak in the suction, or the suction pipe is obstructed.

Sweating of Hydraulic Machinery is prevented by E. Jersey Water Co., Paterson, N. J., as follows: Clean the iron surface with wire brush; give two coats of red lead, then apply heavy coat of litharge and put on as many cork chips as will stick. Apply second coat of litharge and again dust on as many cork chips as will stick.—(E. N., Feb. 11, 1915.)

Pumping Hot Water. Water at high temperature cannot be raised far by suction; supply should gravitate to pump, if water is hot enough to liberate steam under a partial vacuum. If, in feeding boilers, hot feed water is to pass through the pump at about boiling point, it is better to use a size larger than would be required if the heating were done after the water leaves the pump.

Lubricants. As a cylinder lubricant, flake graphite is used alone or with oils. It fills pores and irregularities of cast iron, and imparts to the surfaces of piston, cylinder, and valves a smooth, dense coating and brilliant polish, without apparent pore or crack. Where cylinders "groan" from insufficient lubrication, application of a little graphite and oil through hand pumps will cure the trouble almost instantly. Where cylinders and valves are scored and cut, flake graphite rapidly fills in and overlays irregularities, restoring surfaces to smoothness. When bearings are grease lubricated, 4 or 5 per cent. flake graphite may be mixed with the grease and fed regularly; 4 to 6 per cent. by weight of graphite well mixed with good machine oil makes a satisfactory proportion, thin enough never to clog the oil ways. One great barrier to adoption of superheated steam has been the difficulty of providing adequate and reliable cylinder lubrication; at these high temperatures flake graphite has proved a reliable remedy for insufficient lubrication. Graphite introduced into a steam cylinder, finds its way to the stuffing boxes and packing, and prevents scoring, fluting, or rusting. Any separator, grease extractor, or settling tank that will remove even a fair percentage of cylinder oil will remove all graphite that goes out with the exhaust. Graphite passing into a separator adheres to the baffle plates; remove and clean them from time to time. For certain types of separators it is convenient to purchase duplicate baffle plates which can be changed in a few minutes and the coated plates cleaned at leisure. Specific gravity of graphite is greater than of oil, therefore it is but a matter of time before graphite mixed with oil settles, no matter how finely pulverized or how heavy the oil. Do not attempt to feed graphite mixed with oil in gravity oil cups or sight-feed lubricators, graphite might settle and clog the feed. Have a special oil can for graphite and oil (a heaping teaspoonful of graphite to a pint of machine or engine oil) and shake thoroughly before apply-

ing. Perfectly dry graphite will feed through a common squirt can; follow up each application with a few drops of oil to carry the graphite into the bearings. In "splash" lubricated engines, graphite may be added directly to the oil in the crank case, a teaspoonful to a quart.

Water-pump plungers should be lubricated with a waterproof graphite grease. It cannot be washed off, is unaffected by fresh, salt, alkaline or acid waters and therefore gives the best service. The graphite in its composition reduces friction to a minimum; it also protects the packing and prolongs its life. For gears, slides and other moving parts, it gives the best results as it is tenacious, lasting and an excellent lubricant.

Setting Steam Slide Valves of Duplex Pumps. Pumps from the best shops are tested under steam; if valves are left as when sent from the works, they will be in proper position. In a duplex pump there are two engines side by side and the valve motion is so arranged that one engine operates the valve of the other. Simplest method of setting valves is to place one engine on its striking point, then by removing steam chest covers of the other engine its valve can be so adjusted as to give an excess of ⅛ in. of full port; the engine is then to be put on its other striking point and valves so adjusted as to give ⅛ in. excess of full port to the other end of the cylinder. To set the valves of the other engine, the same operations will have to be followed. The object in giving ⅛ in. excess is to ensure full port opening when the engine is at working stroke, which is a little short of the striking points. Small brass valves at each end of steam cylinders of larger sizes are cushion valves for controlling length of stroke and smoothness of reversal, which ensures silent working and freedom from shock. To properly adjust these valves the pump must be running at its regular work. The valves should all be opened wide. With the valves in this condition the steam pistons should be striking on the cylinder heads; if they do not, they are prevented by some undue friction, caused possibly by the packing being too tight, which should be corrected before going further. If the engine is compound, first try to obtain control by the high-pressure cushion valves; if they fail to give the required control, the low-pressure valves should be used, otherwise they should be left open. Objects to be accomplished are to adjust reversals so that piston travel will be equally distant from each striking point and give proper length of stroke, also quiet working of the pump end.

Compound Pumps: *Cross-compound.* For service against reservoir or standpipe head, or where but slight increase of pressure is necessary for fire service, a cross-compound pump is simpler, and as a rule less expensive than the three-cylinder type; it is also suitable for elevator service under moderate steam pressures or where the service is sufficiently constant to make self-starting unnecessary.

Triplex Compound Pump. Essential feature is the distribution of the low-pressure load between two cylinders and frames, instead of concentrating on one, as in the cross-compound. Where the latter is to be used on a steady load cylinders can be so proportioned as to throw on each of the high- and low-pressure frames, respectively, approximately equal loads. Where there is a wide variation between domestic and fire pressure, advantage of three-cylinder

arrangement is marked; live steam may be admitted to the receiver without danger, since the load, which in a cross-compound would come on one frame, is distributed on two, each as strong as the high-pressure frame, so that no matter how hurriedly or unskillfully the machine is handled while pushing the service up to fire pressure, chances of damage at this critical time are reduced a half. In a duplex double-acting fly-wheel machine the minimum flow is 54 per cent. of the maximum, while in a three-cylinder machine the minimum is approximately 74 per cent. of the maximum.

CYLINDER DATA FOR ENGINES AND PUMPS

Pump Cylinder Capacity. (For table of cylinder capacities, see p. 630.) *Piston Speed.* The ordinary speed for single-stroke pumps is 100 ft. of piston travel per min.; for double-stroke pumps, 140 ft. For feeding boilers, speed should not exceed 50 ft., or 30 to 40 strokes per min. when the boiler is evaporating at its normal rating, 30 lbs. of water per hp. per hr. In fire pumps, where the largest quantity is required, the speed may exceed 200 ft. per min.

Table 144. Strokes Required to Reach Piston Speed of 100 ft. per Min.

Length of stroke, in	Number of strokes	Length of stroke, in	Number of strokes	Length of stroke, in	Number of strokes
4	300	12	100	24	50
5	240	14	86	26	46
6	200	16	75	28	43
7	172	18	67	30	40
8	150	20	60	36	33
10	120	22	55	40	30

Theoretical Discharge. To find the number of gallons delivered per min. by a single-acting pump at 100 ft. piston speed per min. square the diam. of the plungers, then multiply by 4 (roughly).

Proportion between Steam and Pump Cylinder is determined by multiplying the given area of the pump cylinder by the resistance on the pump in lbs. per sq. in., and dividing the product by the available pressure of steam in lbs. per sq. in. The product equals the area of the steam cylinder. To this must be added an extra area to overcome the friction, usually taken at 25 per cent.

Rule for Steam Pressure Required, when the lift, area of the water cylinder and area of the steam cylinder are known: Take half the lift in feet, multiply by the area of the water cylinder in square inches, add 25 per cent. of this to itself and divide by the area of the steam cylinder in square inches (approximate).

Slip of Pumping Engines is so often undiscovered, so important and so easily detected that there is no excuse for the waterworks superintendent to be ignorant regarding the condition of his pumps in this respect. A Venturi meter on the discharge and a counter on the engine are most convenient and all that are necessary for this determination. A comparison of the volumes

pumped, as indicated by each, gives the information desired.—(Kenneth Allen, Proc. Mun. Engrs of N. Y., 1911.)

Cause of "Slip" has not been fully understood and its extent has been underestimated. Old types of reciprocating pumping engines had short water pistons in long cylinders and there was little or no provision to compensate for wear due to sand and grit; piston speeds were low and a slip of 50 per cent. around water pistons was not unusual, which could be proved by closing the discharge gate-valve while the pump was running, when the speed of the pump would not be reduced to less than one-half normal speed. In later types rotative and plunger speeds are much higher and long plungers are used instead of short water pistons, frequently "outside packed." "Slip" past the plunger is generally negligible, but "slip" at the rubber pump valves is often large, due to tardy closing, excessive wear, sticking and clogging of these valves. Many believe that excessive "slip" is always accompanied by noise in the water end of the pump, but noise is sometimes present and sometimes absent. If the trouble is due to a relatively *small* number of *bad* valves there will be noise, but if it results from a *large* number of only *moderately worn* valves, noise can seldom be heard. Sometimes engineers test for "slip" by opening the hand holes underneath the valve decks and allowing full water pressure to be exerted upon the tops of the valves. When they find only a small leakage, they erroneously conclude that "slip" at this point is negligible. In centrifugal pumps "slip" occurs in other ways. "Slip" is now considered to be the percentage difference between the theoretical discharge and the actual volume of water delivered. With crank and fly-wheel pumps short stroking is negligible, but with direct-acting pumps short stroking frequently exceeds 5 per cent. (Builders Iron Foundry.)

In statistics of water unaccounted for, an important cause of inaccuracy is failure to allow sufficient "slip" of pumps. Probably few pumps operate with less than 4 to 5 per cent. "slip" and it is not unusual to find 10 to 30 per cent. Maj. Gillette and J. C. McLennon in report on water waste in Philadelphia, 1906, place average "slip" at 25 per cent. and maximum at 56 per cent., reducing apparent consumption from 233 gals. per capita to 164. Philips found "slip" in Chicago 10 to 28 per cent., which he was able to reduce, by packing and repairing, to about 5 per cent. Evansville, Ind., 1903, cut its apparent consumption from 200 to 107 gals. per capita by repairing pumps.—(E. G. Bradbury, Ohio Eng. Soc., 1912) See also page 458.

STEAM TURBINES

(By J Howard Williams, Mech Engr, Board of Water Supply, New York)

The introduction of modern centrifugal pumps and development of steam turbines occurred almost simultaneously, and the possibility of combining the two in a simple, efficient and inexpensive pumping unit was readily seen. Owing to the fact that the economical speed of steam turbines is higher relatively than that of centrifugal pumps, the combination of the two was delayed materially. Early attempts at driving centrifugal pumps by steam turbines were not successful because of inability to get perfectly manufactured reduction gears. This difficulty has been overcome and efficient gears can be

obtained from several manufacturers, of such design that their life may be considered as long as that of either pump or turbine.

Steam Turbine Types. There are four types of turbines in general use: single-stage impulse, compound multi-stage impulse, reaction, and impulse-reaction. The first type, now obsolete, consisted of a single high-speed runner with a number of nozzles in which entire expansion of steam from full pressure volume to vacuum volume took place. On issuing from the nozzles, the steam impinged upon moving buckets, which checked and reversed the velocity of the current. The reluctance of the steam current to having its direction and velocity altered gives rise to a force against the buckets, which sets the rotor in motion.

Compound impulse turbines are essentially the same as the single-stage, except that steam expansion takes place only partially in the first stage and is continued in successive stages until final vacuum volume is attained. In the reaction turbine, approximately one-half of the expansion in any stage takes place in the stationary buckets, imparting to the steam a velocity substantially equal to that of the moving buckets, so that it enters them without impact.

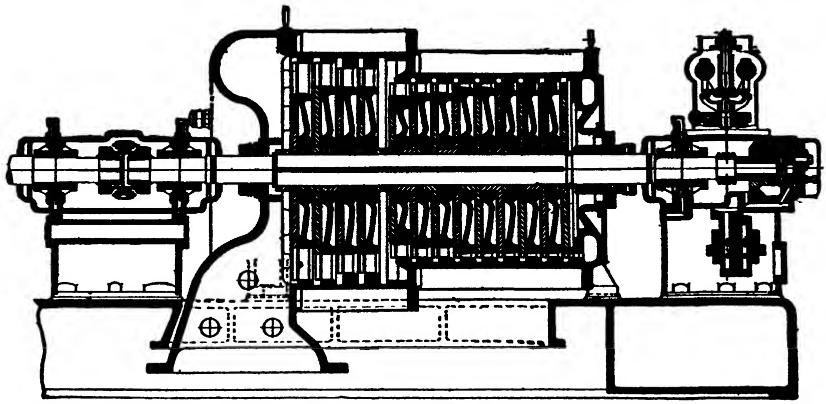


Fig. 274.—Section through 500-kw. steam turbine.

Further expansion takes place in the moving buckets, the spaces between which gradually grow smaller from the inlet to the exit side, forming a ring of moving nozzles. Velocity imparted to the steam by expansion in the moving buckets produces a reactive effort on these buckets, which turns the rotor. This effect is similar to that produced by water issuing from a hose nozzle.

In the impulse-reaction turbine, steam is expanded in nozzles and discharged against a portion of the periphery of the impulse wheel. The intermediate and low-pressure stages are the same as in the ordinary reaction type. Substitution of the impulse element for the high-pressure section of reaction blading is made only in high-pressure stages of machines in which blades are short. The reason for this is that the clearance at the end of the unshrouded blading represents a relatively large ratio to the area through the blades, and it has been found impracticable to use shroud rings on buckets of small section. The unshrouded blades allow steam to spill over the ends, which, in the case of short buckets, may be an excessive waste.

For waterworks all types are used with the exception of the complete reaction machine, which on account of its nature is required to be of large size and power.

Figure 274 shows a turbine manufactured by the Kerr Turbine Co.* of the multi-stage impulse type, steam being received from the governor into an annular space shown on the right. To the casing is bolted a nozzle plate which projects the issuing steam upon the rotor attached to the shaft. The steam is then in turn projected to other nozzles for lower pressure until the last section of the rotor is traversed and final vacuum volume attained. In the machine illustrated, each stage is a complete turbine within itself, sections being added or left out as conditions require. Each section is united to the adjacent one by means of a ground joint, and long bolts extending from end to end hold the entire casing together. Adjacent wheels are separated by a nozzle stage, having self-lubricating carbon packings which insure little friction

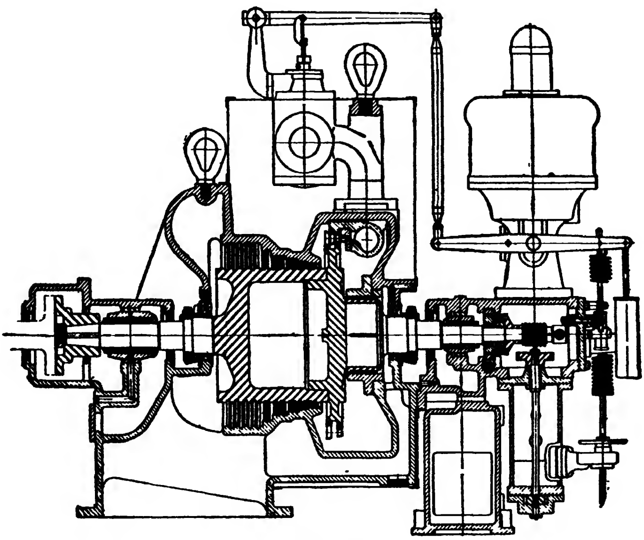


Fig. 275.

and minimum leakage. In Kerr turbines up to 500 hp. it is customary to make nozzle diaphragms in one piece each; above that size they are customarily split in a horizontal plane so that all stationary parts above this plane can be lifted off, completely exposing the rotor.

Figure 275 shows the Westinghouse impulse-reaction turbine. In this machine steam is admitted from the governor to expanding nozzles which extend through a portion of the circumference. From these nozzles the steam passes through rotating buckets, then to stationary buckets and again to rotating buckets, after which it enters a space extending entirely around the machine, from which it enters the first stage of the reaction blading. Expansion takes place through a sufficient number of stationary and moving buckets until final vacuum volume is attained. In the machine illustrated,

* Wellsville, N. Y.

labyrinth packing is used between the intermediate pressure and the vacuum for the purpose of reducing steam pressure on the stuffing box at the entrance end of the machine. The casing of this machine is divided so that all running parts can be exposed by lifting off the cover. This principle is used in the manufacture of most turbines.

Stuffing Boxes. Various metallic and fibrous packings which give excellent results for reciprocating rods are not satisfactory when applied to stuffing boxes on steam turbines. Packing in a steam turbine is subjected to a low pressure, the difference between pressure of the atmosphere and exhaust from the turbine, or some intermediate pressure slightly above atmosphere and atmospheric pressure. As a rule, instead of preventing escape of steam from the turbine, the office of the packing is to prevent air leaking into it. This is usually done by the use of labyrinth packing, into which is admitted a jet of steam, or preferably water, of sufficient pressure and volume to occupy that space to the exclusion of air.

Bearings consist of light shells, babbitt lined, as a rule divided in half so they can be readily taken out and replaced in duplicate. In the case of some very high-speed machines the shells are encased in cylinders, between which there are films of oil to give sufficient movement to nullify the effect of vibration of the shaft. Some bearings are supported in self-alining boxes while others are not. All bearings should be so made that their removal can be accomplished by dismantling as little of the machine as possible.

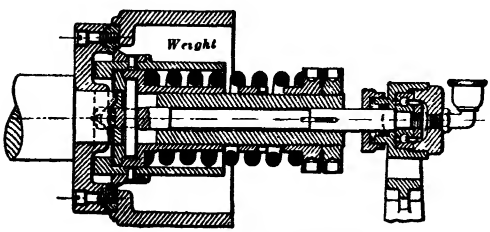


FIG. 276.—Shaft governor.

Thrust Bearings. In some machines theoretical balance of thrust is obtained, but in order to space the rotating members properly and keep them from rubbing, a babbitt-lined thrust is used. Figure 275 shows an unbalanced type; thrust bearing in this case is of the Kingsbury type, which consists essentially of a number of pivoted slippers rotating upon films of oil between them and the fixed member identically as a hydroplane lifts upon and moves along the surface of water. Contact between the slippers and thrust collar is impossible without bringing the machine to rest, as the action of the face of each slipper is such as to entrap a perceptibly thick body of oil and maintain it at sufficiently high hydrostatic pressure to keep the two surfaces apart. Other manufacturers use the marine, multiple-collar thrusts. Satisfactory service is obtainable from any of them.

Of Governors four types are in general use: the shaft governor for small turbines, the geared governor for turbines of intermediate sizes, the oil-relay governors and steam-relay governors for larger sizes. Figure 276 represents the shaft governor. This consists essentially of two weights supported upon hard steel knife-edges, the weights being held in position and adjusted by means of a spring. As the weights move out by centrifugal force, motion is imparted to a double poppet valve by means of a lever directly connected to each of these

members. The governor shown in Fig. 277 is in principle the same as the shaft governor except that it is of lower speed, its spindle being geared to, and driven by, the turbine shaft through a worm and gear. The weights are essentially the same as for the shaft governor, but of heavier construction. Motion is imparted to a double poppet throttle valve in the same manner as is the motion of the shaft governor. Figure 278 shows the oil-relay governor. Operation is similar to that shown in Fig. 277 except that the double poppet valves, in sizes which render this type of governor necessary, are too large to be actuated by weights of ordinary sizes. Consequently, governor motion is imparted to a relay oil valve which admits pressure to either side of a piston which in turn imparts its motion to the governor valve This type may contain not only the main governor valve but an auxiliary which admits steam to inter-

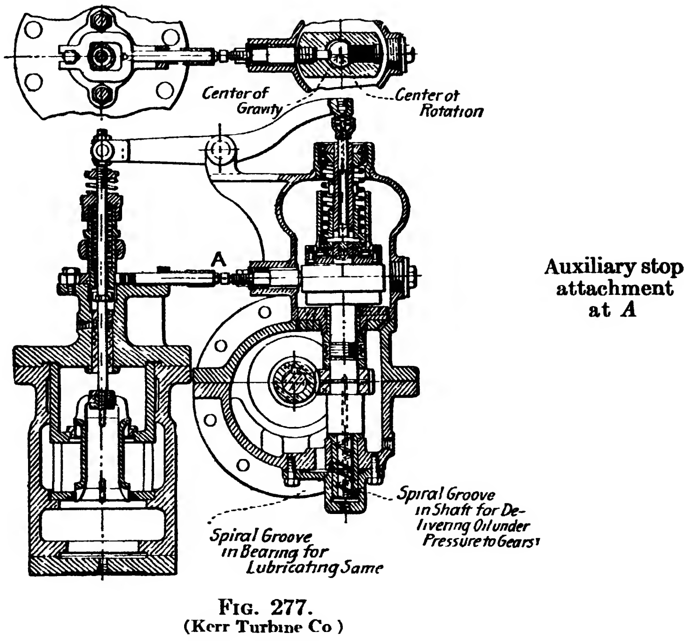


FIG. 277.
(Kerr Turbine Co)

mediate stages in case of overload.* Oil pressure for actuating the governor is obtained from a gear type pump driven directly by the governor spindle. Steam-relay governors are similar to oil-relay governors except that steam is utilized for actuating force. The oil governor has the advantage in that oil is used not only for governing system, but also for the lubricating system, and in case the governor should cease to act from failure of oil supply, the turbine will stop before lubrication of its bearings ceases.

In all three types of governors, there are auxiliary stop attachments, as illustrated in Fig. 277, utilizing weights, the centrifugal action of which is set for higher speed than the governing weights. When dangerous speed is attained, these close the governor valve. In turbines of mixed flow † the principle of the governors is the same as in Fig. 278, a secondary valve (not shown) being util-

* The action of the second valve is through a slotted link which actuates it in case of excessive movement of the governing mechanism

† See footnote, page 481

ized to admit live steam to a high-pressure rotor when the low-pressure supply is not sufficient to carry the load, the high-pressure portion ordinarily running idle. All types are, as a rule, fitted with an outside spring, the tension of which can be changed and speed of turbine modified without stopping it. This is shown on the extreme left of Fig. 278.

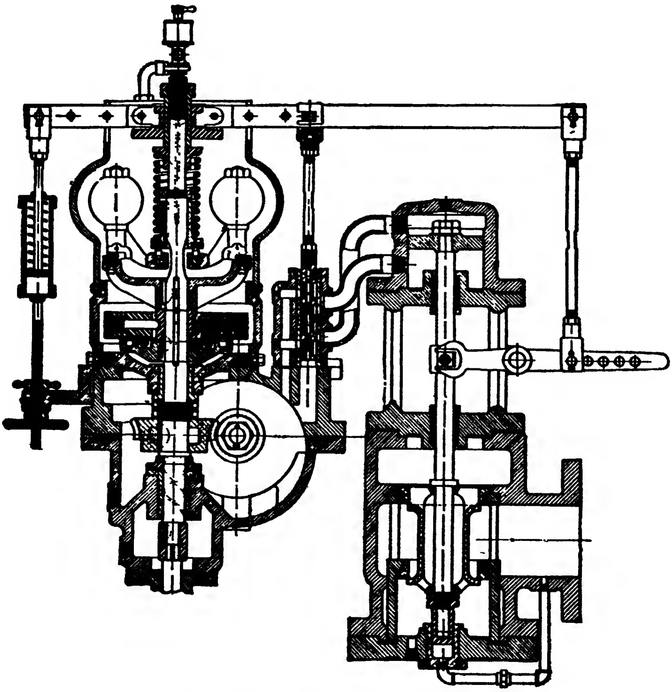


Fig. 278.—Oil relay governor.
(Kerr Turbine Co.)

Reduction Gears used by all turbine manufacturers are essentially the same, varying only in the methods of manufacture and degrees of refinement in manufacture and design. These are in all cases of double-helical type, better known as herring-bone gears. This part was for many years the stumbling block in the way of adapting steam turbines to pumping, but refinements of manufacture have resulted in gears as durable as the turbine itself. The efficiency of this type of gear will vary from 96 per cent. to 98 per cent. when in good condition.

Lubrication. Steam turbines are lubricated in two ways: by oil rings and by forced feed accompanied with ring oiling. In the case of forced lubrication a geared pump is driven from some revolving member. In the case of the oil-relay governors this pump is directly attached to the governor spindle, and oil not required in the governor is relieved through reducing valves and then conducted to the bearings, in some cases at different pressures for different parts of the system. Each bearing is made with an oil well of large capacity, the overflow of which is free and such as not to disturb the main body of oil and prevent sedimentation. Oil overflow is returned to the pump through a

water-cooling system and a screen to separate injurious solids. The principle of simple oil ring lubrication is identical, except that oil is not circulated. Reduction gears are lubricated in two ways, by submersion in oil so that the gears themselves act as gear pumps forcing oil between the teeth, and by means of spray nozzles projecting oil on the pitch line of the teeth at about 5-lb. pressure. Bearings of reduction gears are lubricated by the two methods described and sometimes by the pressure incidental to revolving the gears in oil, which may be as high as 40 lbs. per sq. in. If introduced properly to the bearings, the oil insures satisfactory lubrication. Oil should meet the following specifications. It should be pure mineral hydro-carbon oil free from (1) tarry, slimy or saponifiable matters, (2) acid, soaps or thickeners, (3) water, (4) dirt, grit or other suspended matter. The specific gravity should be between 0.860 and 0.880 at 60° F. Flashing point, with open cup tester, should not be below 334° F. Viscosity as determined by the Saybolt 40° C. Viscosimeter (Standard Oil Co.) should be not greater than 228 sec.

Couplings, both rigid and flexible, are used between turbines and reduction gearing and between reduction gearing and pump. Often the pump has only one bearing, the inboard bearing being that of the gearing. These couplings consist essentially of steel flanges, machined all over, either bolted and keyed on taper portions of the shaft or else solidly forged on the shaft. To prevent accidents, they should in all cases be smooth on the exterior and free from projections. Flexible couplings furnished by most makers are essentially the same. The driving portion carries pins projecting into metal bushings imbedded in some flexible material in the driven flange. Another form of flexible coupling uses driving pins made up of thin plates pinned together at the ends and fastened into pockets.

Steam Pressure. Steam turbines are readily adaptable to a greater range of steam pressures and vacuum than reciprocating engines of whatever degree of refinement. They also utilize superheat of the highest degree yet obtainable. They will work economically with steam as low as atmospheric pressure or as high as practically available. For general waterworks practice 175 lbs. pressure, 100° superheat and 28½- to 29-in. vacuum are recognized as ideal. Higher pressures and superheat are obtainable only by sacrificing life of boiler plant. Lower pressures result in decreased efficiency as pressure decreases. Many turbines for low pressure, of mixed-flow type,* have been installed and have been generally recognized as a source of economy. This point is disputed by some manufacturers, who claim it is usually cheaper to discard the steam engine, exhaust from which is used in the turbine, and purchase a turbine using full boiler pressure. This is, however, a point requiring investigation for each case. Foundations, condensing apparatus and other items, exclusive of maintenance of steam engines, often bring the cost to as much as or more than the cost of high-pressure turbines. The objection to mixed-flow turbines is the inefficiency resulting from the attempt to make a machine for two pressures, with the result that during a large part of the time one portion is running idle. Economy for given conditions is essentially the same by all manufacturers.

* Low-pressure steam is admitted to intermediate stage from some exterior source, such as steam-engine exhaust.

Table 145. Consumption of Steam per Brake hp.-hr. by Steam Turbines*

Brake Hp.	150 lbs. dry steam, 28" vac., lbs. per hour	100 lbs. dry steam, 28" vac., lbs. per hour
800	13.2	14 0
700	13.4	14.2
600	13.5	14 4
500	14.0	14.8
400	14 6	15 5
300	15 1	16.2
200	15 7	16.9
100	16 5	17 6

For 27-in. vacuum instead of 28 in., above steam consumptions would be increased 5 per cent. At ¾ load, steam consumptions would be increased 4 per cent.; at ½ load, 15 per cent. Another prominent manufacturer gives the following rates and corrections for 125 lbs. pressure, dry steam, and 28½-in. vacuum.

Brake hp	Steam consumption, lbs per Brake hp.-hr.
1500	12 1
1000	12 5
750	13 0
500	13 5
250	15 0

Correction for steam pressure is 1½ per cent. for each 10 lbs. between 100 and 125 lbs.; correction for vacuum is 5 per cent. per in. between 26 and 29 in.; superheat, 1 per cent. for each 10° up to 100° F.; for pressures above 125 lbs. correction will be 1 per cent. for each 10 lbs. pressure, for vacuum and superheated steam as above stated.

Economy. Duty of turbine pumping units has almost as wide a range as that of reciprocating units. Duties as high as 140,000,000 ft.-lbs. per 1000 lbs. of steam are not uncommon, and 152,000,000 ft.-lbs. have been obtained. Following are data and specifications for a steam-turbine-driven pump for Montclair Water Co., Paterson, N. J., installed in 1915. The turbine, shown in Fig. 275, is 1000 hp., reduction gearing 3900 to 1400 r.p.m., driving a 12-mgd., 18-in. double-suction centrifugal pump against 310 ft. combined suction and discharge head, measured adjacent to suction and discharge nozzles. The condenser is in a by-pass and requires 1150 g.p.m. circulating water obtained directly from pump. Air and condenser pumps are direct-connected to the main pump shaft. Duties in this table, therefore, include power consumed by auxiliaries. Steam pressure is 175 lbs.; superheat at throttle, 100° F.; floor space occupied 19 ft. by 6 ft. 9 in., hight 7 ft.; capacity at 150 ft. total head, 17 mgd.

Water discharge in gals. per 24 hr.	Temp of water through condenser, ° F.	Vac. in condenser, inches	Guaranteed duty, ft -lbs. per 1000 lbs. steam supplied to turbine
12,000,000	55	29 00	144,000,000
12,000,000	65	28 60	141,000,000
12,000,000	70	28 35	139,000,000
12,000,000	75	28 10	137,000,000

* Information furnished by a reputable manufacturer.

Philadelphia has installed several pumping units which vary in efficiency from 70 to 150 million ft.-lbs. per 1000 lbs. of steam. Throughout the U. S. large cities either have such units, or are installing them in preference to reciprocating units. A steam turbine requires only a small building, light foundations and light crane service. Costs of repairs, attendance and operation are low. The oiler attendant may be dispensed with, as oiling is automatic. A less efficient engine man may be employed, with consequent saving in wages. No exact information is obtainable as to the amount of oil used; a machine of 1000 hp., however, can be relied upon not to use more than one barrel of good oil per annum. This quantity is required chiefly because of removing oil to insure cleanliness, rather than for replenishing. Oil, after being removed from turbine bearings, can be used for other purposes. It is the consensus of opinion that 2 per cent. of original cost is ample allowance for repairs. Amortization has been high, for the reason that development has been rapid. Although improvements are still being made, economy has nearly reached its maximum, so that turbines of the most economical types at present obtainable will not need to be replaced by more efficient machines. Amortization is computed by some as high as 8 per cent. on new machines, while others claim 5 or 6 per cent. is more nearly right.

Cost. Competition results in practically the same price per horse-power from all manufacturers. Condensing, geared turbines may be computed on the following basis, with installation extra: 250 brake hp., $15; 500 brake hp., $11; 750 brake hp., $9; 1000 to 2000 brake hp., $8 per brake hp. Auxiliaries cost installed, approximately as follows per main brake hp.: 250 brake hp. of turbine, $4; 500 brake hp. of turbine, $2.50; 750 brake hp. of turbine, $2; 1000 to 2000 brake hp. of turbine, $1.50 to $1.

Table 146. Some Steam Turbine Pumping Plants in Use, 1915

Place		Delivery, gals. per day	Ft head
Pittsburgh, Pa.	1 unit	100,000,000	56
Cleveland, O.	1 unit	30,000,000	260
Cleveland, O.	2 units	100,000,000	50
		60,000,000	50
Omaha, Neb.	1 unt	30,000,000	90
Lynn, Mass.	15,000,000	13,000,000	134
	Combination pump	2,000,000	244
Indianapolis, Ind.	1 unit	6,000,000	338
Philadelphia, Pa	2 units	20,000,000	330
Toronto, Can.	2 units	24,000,000†	270
Toronto, Can.	1 unit	24,000,000†	60
Toronto, Can.	1 unit‡	20,000,000†	270
Toronto, Can.	1 unit	10,000,000†	270
Montreal, Can.	1 unit	36,000,000†	206
San Antonio, Tex.	2 units	6,500,000	273
St. Louis, Mo.	1 unit	20,000,000	....
Youngstown, O.	1 unit	8,640,000	335
Youngstown, O.	3 units	8,640,000	350
New Haven Water Co.	1 unit	15,000,000	125
So. Pittsburgh Water Co.	1 unit*	8,000,000	407 5

Turbines at Rock Island, Ill., Atlantic City, N. J., and Bay City, Mich., also.

* Guaranteed duty, including auxiliaries, 100 million ft.-lbs. per 1000 lbs. steam at 120 lbs.; no superheat, 28-in. vacuum. † Imperial gals., see page 623.

‡ Guaranteed duty, 118 million ft. lbs.; duty during test, over 130.

CENTRIFUGAL PUMPS *

(By J. Howard Williams, Mech. Engr., Board of Water Supply, New York)

Development. Until very recent years, centrifugal pumps had little usefulness in waterworks, except as auxiliaries, because of the inability of manufacturers to produce an efficient source of power for driving them. As main pumping units driven by steam engines, they proved failures, because of the impossibility of building economical steam engines of the speeds required.

Modern electric motors increased the usefulness of centrifugal pumps, but even with motors they can be used as main pumps only where electric power is available at low cost, or where service is too intermittent to warrant maintaining a steam plant. For fire service, or in special cases, such as unwatering mines and tunnels, the electrically driven pump has a field. With the advent

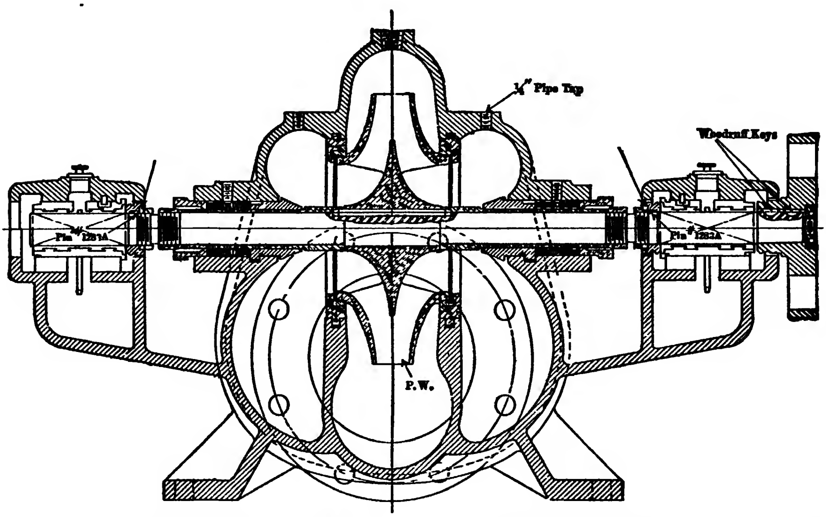


Fig. 279.—Typical section of De Laval single-stage, double-suction centrifugal pumps of small and moderate size.

of the steam turbine, centrifugal pumps really came into usefulness as main pumping units as well as auxiliaries, and although the steam turbine is a later machine in development, it may be said that efficient electric and steam turbine drives were developed simultaneously. (For the steam turbine, see p. 475.) In a few cases water turbines are being used as prime movers for centrifugal pumps, and as such are proving economical.

Centrifugal Pump Types. There are two and they may be subdivided into classes according to details of construction. In the principal type the impellers discharge water from their peripheries into collecting chambers, in which the velocity of the water is slowed to that of pump discharge. In the second type impellers discharge into chambers having guide vanes which conduct the water to collecting chambers at their peripheries, and at the same time bring the velocity to that of discharge in such a way as to prevent im-

* For data on testing apparatus, see Merriman's "Treatise on Hydraulics," John Wiley & Sons, 1912.

pact and eddies and to convert kinetic head into static head with minimum loss. Both can be had in single or multi-stage or single-stage multi-impeller. Pumps of the first type are known as centrifugal pumps, those of the second as turbine pumps. Single-stage centrifugals are made in two general classes. In one class, end plates carrying bearings and pedestals are bolted to the body

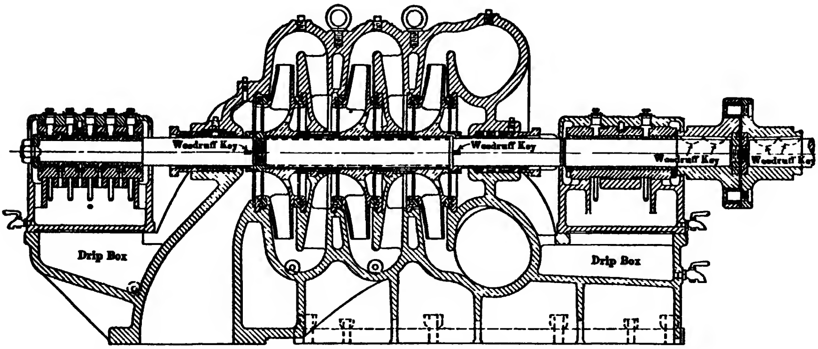


FIG. 280.—Typical section of De Laval multi-stage centrifugal pump.

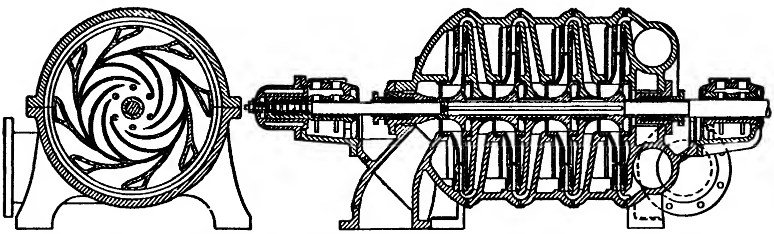


FIG. 281.—Sectional elevation of 4-stage, type "S" turbine pump. (Alberger Pump Co.)

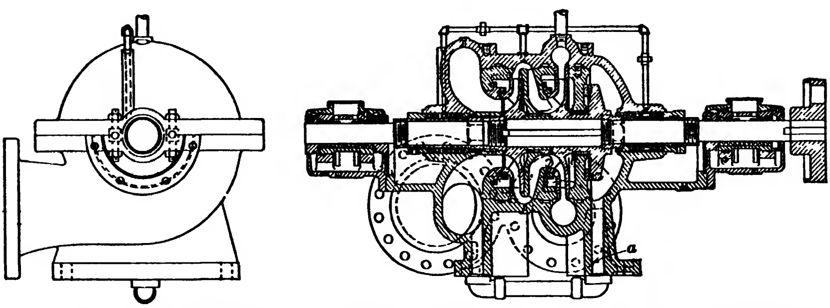


FIG. 282.—Worthington 8-inch, 2-stage turbine pump, Underwriter Fire Service. a, non-throttling orifice.

of the pump, which houses the impeller and forms the discharge chamber; this class is made with single or double suction. The other class is divided horizontally on the center line plane with all pipe connections and shaft bearings in the lower half, thereby permitting access to the running parts without removing more than the top half of casing. Pumps 3 in. and smaller are gen-

erally made with end plates, as in such sizes they can be as easily dismantled as split casings. Multi-stage centrifugal pumps are made along the same lines, except that only single suction can be had. Turbine pumps are made in both classes. Single-stage multi-impeller pumps are made only in the split-casing class.

Impellers should always be bronze so as to avoid corrosion; gun metal (88 copper, 10 tin, 2 zinc) is recommended. In order to obtain best results by reducing skin friction and surface eddying, impellers should be machined both inside and out. Inaccessible surfaces on the interior should be scraped and

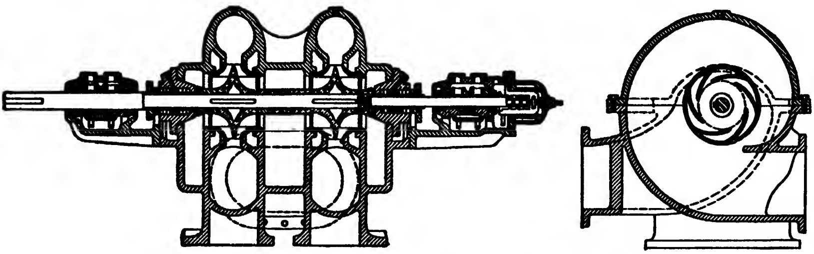


FIG. 283.—Sectional elevation of two-impeller volute pump. (Alberger Pump Co.)

draw-filed to as high a polish as practicable. Impellers should be statically balanced by testing with knife edges, and their centrifugal balance made sure by idle rotation at rated speed. Impellers are made in two types: with only one inlet and with inlet on both sides; in latter type the middle of impeller is carried up in the shape of a wall to afford easy passage to the water. Sides of impellers should always be closed to avoid friction of water with pump casing, and to afford an actuating force to all of the water by eliminating clearance at side of vanes. Vanes should always be fitted to templets to insure accuracy of angle and balance.

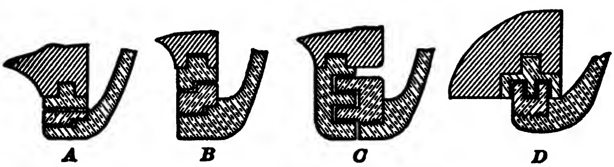


FIG. 284.

Impeller Packing Rings. Fig. 284 shows the four types in common use. Economical results are obtainable from any and all of them. All packing rings should be hard bronze. One prominent manufacturer uses a composition (5 parts tin, 1 part copper) which is said to have been selected after careful experiment.

Shafts. Impeller shafts should be steel, if composition of water to be pumped so permits, or bronze if the water is salty, acidulous or alkaline. Steel shafts should be bronze-jacketed in all cases for protection against corrosion and to permit repair of worn or scored parts; they should be of

high strength steel to stand the stress, and especially to afford rigidity in pumps of high rotation. Shafts and shaft jackets should be provided with straight keys to rotate impellers; should be so made that a nut holds impeller against a shoulder to prevent impeller from drifting along the shaft; should have collars near ends of bearings to clear off oil by centrifugal force and prevent it from creeping out of bearing and wasting.

Couplings. For shaft couplings see p. 481.

Bearings. Outside bearings as a rule are of the shell type similar to the bearings described on p. 478 for steam turbines, except that forced lubrication is seldom used. To be reliable the pump should have bearing sleeves easily removable without disturbing the other parts of the pump, as it may become necessary many times in the life of a pump to renew worn bearings. All oil bearings should have ring oilers and deep wells to permit sedimentation. Interior bearings of hard bronze are placed between impellers and at bottoms of stuffing-boxes; these are water lubricated only. Small pumps and large low-speed pumps are frequently made with lignum vitæ bearings with water lubrication; they should be avoided as they have proved unsatisfactory in service. The quality of oil required is the same as for steam turbine, see p. 481.

Diffusion Vanes. Manufacture of pumps of the turbine type is declining for many reasons, the principal being that diffusion rings increase the diameter, resulting in the pumps being heavier and therefore not on a competitive commercial basis with pumps of the centrifugal type. They do not lend themselves as readily to work under varying heads, as the efficiency is greatest when at one particular head, and any variation from that head causes a more rapid decrease in efficiency than in centrifugal type pumps. Diffusion vanes are liable to wear for two reasons, grit in water at high velocity, and eddies at point nearest the impeller. No pump can be designed without some impact. Impact upon the point of the diffusion vane causes pitting and wear even from clean water, so that in the course of time diffusion vanes may become an economical detriment rather than a benefit to the pump. Turbine type is the most efficient, if properly proportioned, and it may be said that cost is the principal objection, although many manufacturers attempt to make the customer believe otherwise, using the arguments above cited. All surfaces of diffusion rings should be finished smooth for the same reason that impellers should be smooth. Diffusion rings should be easily removable. Pumps not split along the center are very likely to rust so as to prevent removal of diffusion rings and guide passages.

Noise in Pumps. Noise in a centrifugal pump is an indication of impact; impact means poor design. Specifications should forbid noise, and the purchaser should require the builder to eliminate it, as it is an indication of wear on impeller and on casing. Noise of impact is a rattling and pounding, but should not be confused with the hum which exists in every machine rotating at high speed. Objectionable noise may be caused by air in the suction line or leakage through stuffing-boxes.

Stuffing-Boxes. Practically but one type of stuffing-box is in use, an ordinary box holding a certain number of turns of square hemp, on top of which

is a lantern gland, and on top of this more hemp. Leak-off is provided opposite the lantern gland so as to permit a certain amount of leakage (which may be wasted or returned to suction side), and thereby avoid tightening the gland to such a degree that heating would develop. In pumps where partial vacuum exists on stuffing-boxes, water at low pressure is admitted to the lantern to fill this space to the exclusion of air. Purchasers should be sure that pumps have this provision. All stuffing-boxes should be provided with a drip receptacle draining leakage to waste. Casings should always be lined with bearing bronze where shaft projects inward from stuffing-boxes.

Thrust Bearings. A centrifugal pump is never wholly balanced against end thrust. Pumps with double suction and single outlet are nearest to this realization, but it is necessary to provide collars on the shaft to keep the impeller in its proper position inside the casing and to take thrust resulting from unequal flow of the two inlets and inequality of areas of passages. Liability to end thrust is greatest in the multi-stage classes. Many unsuccessful

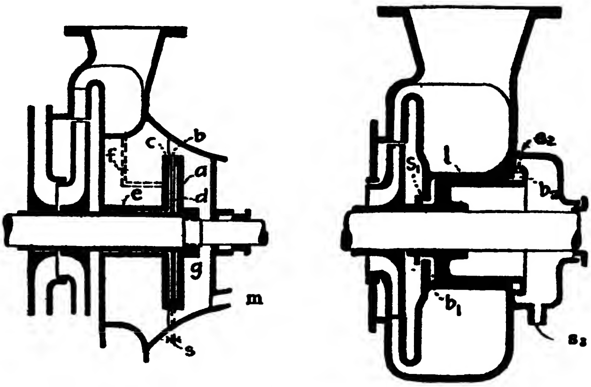


FIG. 285.—Two types of water thrust bearings.
f and s_1 = 1st orifice; *b* and s_2 = 2nd orifice; *m* and s_3 = 3rd orifice; *a* = stationary piston; *d* = rotating piston

attempts have been made to balance against end thrust, and although it can be done theoretically, it is never actually done. The latest development in theoretically balanced multi-stage pumps, is the design of impellers with baffle rings front and back, with holes in the back of each impeller so as to equalize the pressures. This remedy is only partially successful as it cannot be used except in conjunction with a thrust bearing. See Fig. 280. To hold the pump parts in correct relation, thrust-bearings of the marine, ball, roller and piston type are all in successful use. In Fig. 281 is shown pump with multi-marine collar thrust. Fig. 282 shows a pump with a balance piston for carrying end thrust. Fig. 285 shows two principal types of water thrust; water is admitted to one side of the piston through an orifice, which may be of fixed or variable size, a second orifice allows the escape of the water through to the back of the piston, and a third orifice allows it to return to some low-pressure part of the pump, usually the first stage. By floating within fixed limits the piston automatically controls pressure on the two sides, and keeps the pump rotating not upon bearings, but upon water. In some types of hydraulic thrusts the third orifice is

not throttling, as in the case of Fig. 282 in which all throttling takes place in the first two orifices. In other pumps the first orifice is not throttling and all throttling takes place in the last two orifices. In all types at least one orifice is between a moving member and some fixed portion of the pump. Such orifices should be bronze-lined for bearing purposes and to prevent scoring resulting from high velocity.

Bed Plates. Bed plates should be heavy, so designed as to support both pump and motor, and fitted with dowels for accurate alinement of pump and motor. When turbines and pumps for large units are purchased from different manufacturers, the bed plate for each is frequently separate; in this case the foundation should be large. Bed plates should be deep and provided with openings in the top through which concrete and grout can be admitted after alinement of the machine upon its foundation. By filling the interior, noise and vibration are materially diminished. Purchaser should always insist on heavy bed plates for waterworks pumping units.

Testing. When the pump is completely machined and erected it should be subjected to a hydrostatic test of not less than 1½ times its maximum discharge pressure for a sufficient period to insure structural strength of its parts. No pump should be accepted without static and operating tests. No manufacturer can guarantee the performance of a pump otherwise than by trial. The purchaser should always have a competent, disinterested hydraulic engineer to witness the tests of a pump where economy is of importance. In obtaining the quantity of water, the nozzle, standard orifice, weir, pitometer, pitot tube, Venturi meter or tank is used. Skill is required in the use of all of these instruments, and the purchaser is warned against deception, as it is easy. The nozzle is probably the most accurate, the standard orifice second, the Venturi meter third, and the weir fourth, with but small differences; pitometers and pitot tubes should not be used as they are much more liable to serious error. When checking the quantity discharged after installation where only the pitometer is available, this instrument should not be used in other than a straight pipe of accurately known diameter at a point at least 50 diam. from the nearest turn upstream, and 20 diam. downstream. All pressure gages should be carefully calibrated in the presence of the witness and allowances made, as no dial pressure gage indicates correctly.* Total head against which the pump works is the algebraic sum of the suction and discharge pressures, corrected for velocity head in suction and discharge pipes. All test data accepted from the manufacturer should be sworn to before a public officer, and even then may be not altogether trustworthy, as the personal prejudices of the manufacturer's agent are always to be considered. No form of meter, other than a Venturi, should be trusted in testing. In testing a pump with a motor, certified characteristic data of the motor should be obtained from the electrical manufacturer, in order to know the power which the motor is delivering to the pump. All volt-, watt- and ammeters should be of the precision type, as commercial instruments may vary as much as 2 per cent. In testing with an alternating-current motor, the power factor of the motor should be used, or else the power should be taken from the watt-meter, watt-meter reading equalling the product of the power and the volt and

* The best method of measuring pressure is by means of mercury column.

ammeter readings. Purchasers are cautioned in regard to efficiency and power factors of motors as they vary with change of load upon the motor, and its temperature. If the pump is to handle warm or hot water, the temperature of the water during test should be as near that of the temperature under service as is possible to obtain. During service, a pump should be carefully tested from time to time, and the results compared with those obtained at the time of purchase, in order to determine the need of examination and renewal of interior parts.

Pump Characteristics. Relation between head, capacity, speed, and horsepower required to drive a centrifugal pump can be expressed by curves plotted either from calculations or from test data, and these curves represent the char-

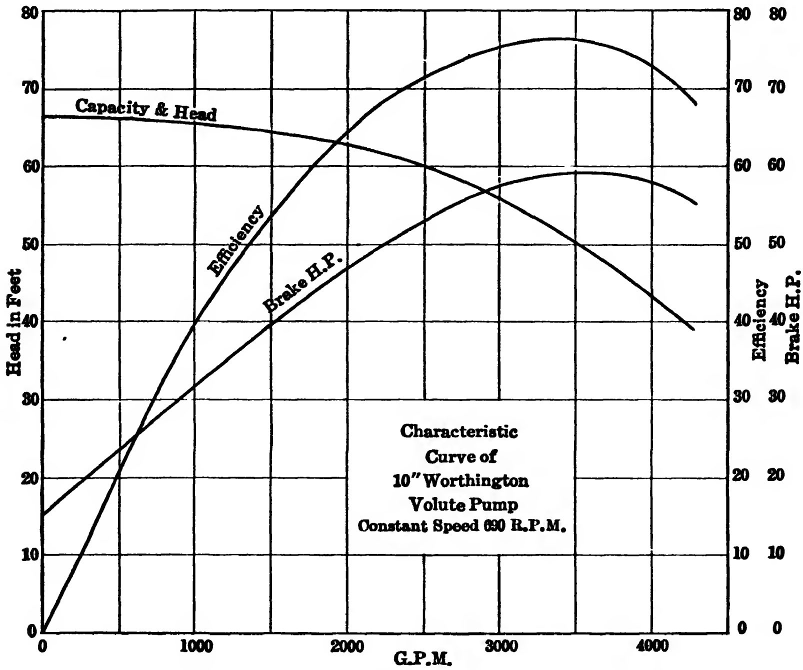


Fig. 286.

acteristics of the particular impeller chosen. Without altering the pump casing to any great extent, any one of a variety of impellers each having separate and distinct characteristics may be used, so that for any given size of pump, capacity may remain constant while head, speed and horsepower required are varied over a wide range. Theoretically the capacity produced by an impeller varies directly as the peripheral speed, while the head produced varies as the square of the speed, and the power required to drive it varies as cube of the speed. Typical characteristic curves are shown in Fig. 286. In these curves, capacity, total head pumped against, brake horsepower and efficiency are all shown when the pump is operating at a constant speed. For instance, the capacity required is 3500 g.p.m.; a vertical line from the point midway between 3000

and 4000 on the bottom scale intersects the curve above; a line from this intersection to right or left, according to whether horsepower and efficiency or head is required, gives total head pumped against as 50.5 ft., brake hp. required to operate as 59, and pump efficiency as 76 per cent. This is not efficiency of the unit, but means that a motor or steam turbine capable of delivering at least 59 brake hp. at the shaft will be required, and that 76 per cent. of this brake horsepower will be converted into water horsepower by the pump. With some impeller designs horsepower required when operating against heads lower than that for which the pump was designed increases so rapidly that serious damage may be done to the driving motor, especially if the motor is of the constant-speed type, while with others it is possible to reduce the operating head to a minimum without overloading the motor beyond the safe limit allowed by motor builders. This condition is met by fire pumps, where variation in number of streams or bursting of hose would be disastrous, unless the impeller were

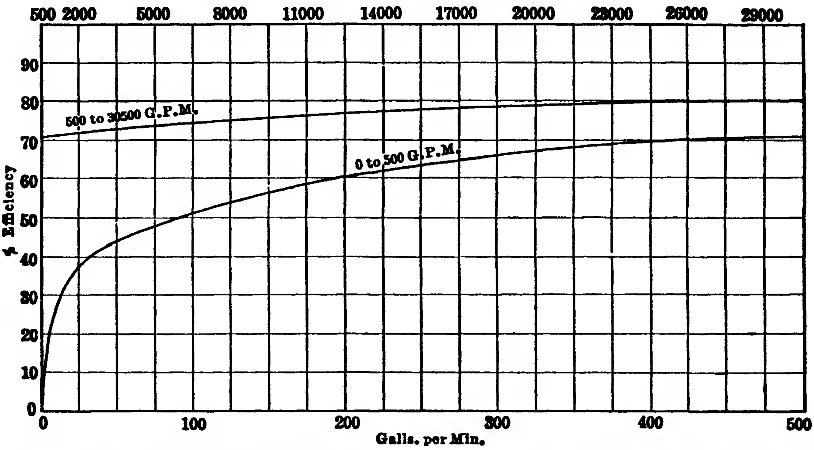


FIG. 287.—Curves showing relation between maximum efficiency and capacity.

designed for such emergencies. Thus it is that while the centrifugal pump has a wide range of usefulness, each impeller is special and great care must be exercised to furnish the pump builder with all important data.

Efficiency of pump is ratio of water horsepower delivered to brake horsepower received; the difference between this and unity is the waste of power taking place in the pump. In Fig. 287 are curves showing relation between maximum efficiency and capacity of centrifugal pumps. As this includes all sizes and quantities it cannot be taken as an average but is true when head and speed are in their best relation. Fig. 288 goes further into detail and shows the relation of quantity, speed and efficiency for 6", 8" and 10" double-suction, single-stage centrifugal pumps. The dividing curve is in each case common to pumps on either side. Each manufacturer of centrifugal pumps has a stock line which is usually not modified for speeds other than that for which their impellers are designed; the customer is therefore furnished a pump which would be more efficient if the speed were better related to head.

Important Data for Centrifugal Pump Designers:

1. Number of pumps required and nature of service.
2. Is vertical or horizontal type of pump desired?
3. Capacity required in g.p.m. (U. S. or Imperial).
4. Total pumping head—suction and discharge head, plus pipe friction.
5. Maximum suction lift and distance from source of supply.
6. Length and size of suction and discharge pipes, giving number and kind of elbows.
7. If pumping head is variable, what is maximum variation for both suction and discharge?
8. If pump is located below source of supply, will it be submerged or placed in a dry pit?

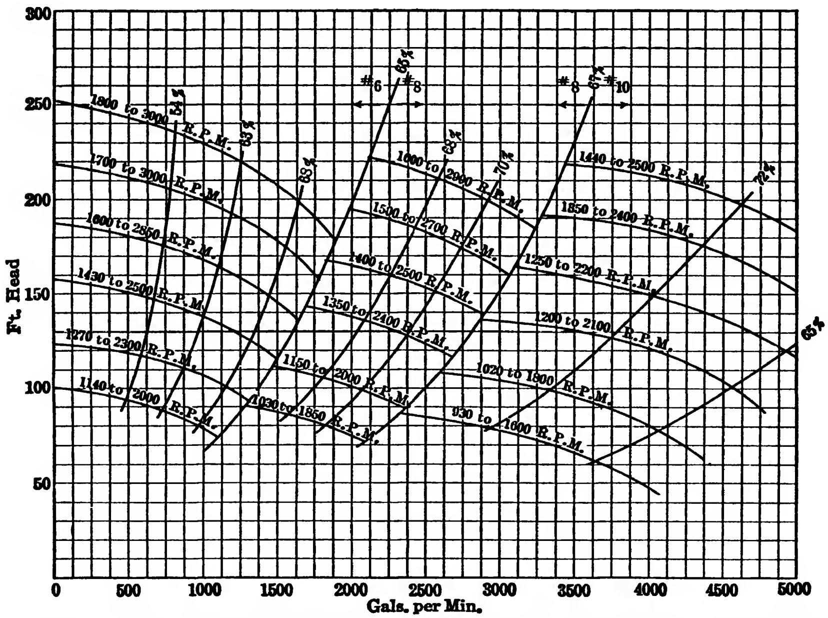


Fig. 288.—Curves showing heads, capacities and efficiencies at various speeds for No. 6, 8 and 10 double-suction, single-stage, split-case centrifugal pumps.

9. Will pump be required to operate continuously? If not, at what intervals?
10. Nature of liquid to be pumped—hot, clear, fresh, alkaline, cold, gritty, salt, or acidulous; give temperature and specific gravity.
11. If pump is to be driven with electric motor, give current characteristics; if direct current, give voltage; if alternating current, give phases, cycles and voltage; give type of electric control desired.
12. If pump is to be steam-driven, is reciprocating engine or steam turbine desired? Give steam pressure, superheat, if any, condensing or non-condensing; if condensing give vacuum.
13. If pump is belt-driven, give speed and diameter of driving pulley, if possible.
14. Give direction of rotation.

Instructions for Setting up and Operating. The pump should be located so that all parts are accessible. Foundations should be of sufficient depth to

Table 147. Sizes, Capacities, Speeds and Powers for Worthington Standard Volute Pumps

Size, discharge diam., in	Capacity gal per min. Normal	Capacity gal per min. Maximum	Speeds in r p m (numbers separated by dash) and horsepowers for given heads in feet: 5 ft.	10 ft	15 ft	20 ft	25 ft	30 ft	35 ft.	40 ft.	45 ft	50 ft.	55 ft	60 ft.	65 ft.
1½	45	60	500–810 ⅓	680–1080 ½	780–1170 ¾	870–1305 ¾	950–1370 1	1020–1550 1¼	1120–1650 1½	1260–1740 1½	1400–1845 1¾	1480–1935 2	1580–2025 2¼	1660–2085 2½	1720–2160 2¾
2	75	100	420–720 ¾	570–925 1	650–1000 1¼	725–1125 1¼	790–1225 1½	870–1325 2	940–1415 2¼	1050–1495 2½	1160–1555 2¾	1230–1655 3¼	1320–1750 3½	1680–1780 3¾	1430–1855 4
2½	125	150	360–630 1	485–825 1¼	560–875 1½	625–980 1¾	680–1080 2¼	740–1160 2½	800–1240 3	900–1305 3½	1000–1385 3¾	1050–1450 4¼	1125–1520 4¾	1190–1565 5	1230–1630 5½
3	200	250	315–540 1½	425–735 1¾	490–780 2	545–875 2½	590–955 3¼	650–1035 3¾	700–1100 4½	790–1160 5	875–1235 5¾	925–1285 6¼	940–1350 7	1040–1395 7½	1080–1450 8¼
4	350	450	280–450 2	375–710 2½	435–765 3½	485–855 4½	525–940 5¼	580–1015 6¼	625–1100 7¼	700–1135 8¼	775–1170 9¼	820–1240 10¼	880–1305 11¼	920–1325 12½	960–1385 13½
5	600	700	250–385 2¾	340–595 4	390–675 5	435–735 5½	475–810 7	520–855 8¼	560–910 9¾	630–955 10¾	700–1010 12¼	735–1050 13¾	790–1080 15¼	830–1115 16¼	860–1160 17¾
6	800	1000	230–315 3¾	310–510 5½	355–575 6½	395–630 7½	430–685 9	475–735 11	510–780 12½	570–820 14½	635–855 16¼	670–890 18	720–925 20	755–955 21½	780–990 23½
8	1500	1800	195–260 5¾	260–420 9	300–475 11½	335–525 13	365–570 16	400–620 20	435–640 23	485–675 26	540–710 29½	570–740 32½	610–765 36	640–800 39	660–820 42
10	2500	2800	180–225 8	245–400 13	280–450 16½	310–490 20	340–525 25	370–555 30	400–585 35	450–620 40	500–640 45½	525–655 50½	565–675 55½	590–700 60½	615–720 65½
12	3500	4000	168–180 11	225–370 17	260–415 22	290–450 29	315–475 37	345–500 44	375–530 50	420–550 58	465–575 65	490–600 78	525–620 80	550–645 87	570–655 94
14	5000	5500	158–170 14½	210–340 23	245–380 30	270–410 40	295–440 50	325–465 60	350–485 70	395–510 79	435–530 89	460–555 99	495–565 109	520–600 119	540–625 129
16	6500	7500	148–160 18	200–325 32	230–365 40	255–395 54	280–425 68	305–445 81	330–465 95	370–495 108	410–515 122	430–530 135	465–550 149	485–570 162	505–585 175
18	8000	9500	134–145 24	180–295 40	205–330 51	230–355 69	250–380 86	275–400 102	295–425 119	330–440 137	365–460 155	390–475 171	415–495 188	435–510 205	450–525 218
20	10000	12000	120–135 30	162–250 50	185–315 65	205–325 86	225–340 108	245–365 130	270–380 150	300–395 172	335–415 193	350–435 215	375–445 236	395–460 258	410–475 280
24	15000	18000	105–125 46	142–235 76	162–265 97	180–290 130	200–310 162	215–325 194	235–340 227	260–355 259	290–370 293	305–380 324	330–400 357	345–410 390	360–425 420

Pumps can be designed for any speeds within the range given in the table, but when a certain speed has been selected and the pump designed accordingly, no change in speed can be made without affecting capacity and head. Powers given are for the maximum quantity of water; power required for less water is approximately in proportion to quantity when quantity is not less than the maximum capacity for next smaller size pump. Motors should be selected of somewhat larger capacity than the horse-power given, to allow for possible overload.

support the pump base rigidly, and should be located in accordance with the foundation plans of the pump, but sufficient allowance should be made for lateral adjustment of the bolts. After the bed plate has been bolted in position, and after the pump and motor have been placed in their final positions, the unit should be carefully tested for alinement by inserting a machinist's thickness gage between the coupling flanges and by making sure that the whole shaft revolves freely when coupled together. The bed plate should then be rigidly grouted in place and the pump and motor dowelled to the bed plate. Suction and discharge piping may then be connected, but care should be exercised that these pipes come to the pump flange evenly and without strain. After pipe connections are made, alinement should again be checked. Care should be taken to see that no air pockets are in the suction line. The pump should be placed close to the water, and the suction pipe should have few or no bends. Wherever possible, suction lift should not exceed 15 ft. from level in well to center of shaft of pump, for cold water; hot water should come to the pump with little or no lift.

Priming Pumps. Most centrifugal pumps do not operate unless fully primed, and it is necessary to use some method of priming. Simplest is to close the valve in the discharge and fill the system, when there is a foot valve on the suction pipe. In many cases, foot valves are not practicable nor desirable, and it is then necessary to close the discharge valve and prime with a vacuum pump or ejector. When foot valves are used, it is necessary to see that they are of the free-opening, clapper type with area through the valve sufficiently great to avoid unnecessary resistance. After vacuum priming, the pump may be brought to rated speed with the discharge valve closed, and then the pump put into service by opening the discharge valve. Air or other gases in the suction pipe or well will reduce the delivery and efficiency. Care should be taken to eliminate air before it enters the suction pipe by using a system of baffling.

Operation. Since a centrifugal pump depends upon speed to deliver the rated capacity against the rated head, this speed should be maintained. Great care should be given the shaft stuffing-boxes, especially those on the suction end, to see that no air enters. When water seal lanterns* are used in stuffing-boxes, the gland bolts should be drawn up tight at first, then released so that they will be finger tight and allow a small leak of water through the gland. This method secures long life for packing and reduces liability of cutting the shaft. So long as the attendant can see water emerging from the stuffing-box, he is safe in assuming that no air is entering. A gate valve and a check valve should be placed in the discharge close to the pump. The latter is especially necessary in long lines, because if the pump is suddenly stopped, the momentum of the long water column may cause a water-hammer that will split the pump casing. Before starting new or unused centrifugal pumps, it is advisable to clean out the bearings thoroughly, including the thrust bearing, by pouring in kerosene and allowing it to run out at the bottom, as dirt is liable to get in during shipment or idleness. The bearings should then be filled as full as possible with first-class lubricating oil similar to turbine oil. In order to discover whether a pump is operating under the conditions for which it was sold very little investigation is required. The dynamic head is readily obtained

* See p. 487, "Stuffing Boxes."

by a vacuum gage on the suction side and a pressure gage on the discharge; both at the pump outlet. The sum of simultaneous readings of these gages plus the vertical distance between their centers gives the dynamic head. Should the water flow to the pump under pressure, this pressure must be deducted from the discharge pressure and then the difference between the centers of the two gages should be added.

Troubles. If on starting a centrifugal pump, after it has been thoroughly primed, it is found that only a small quantity is discharged or none at all, air is probably present due to defective suction pipe joints or excessive suction lift. Should the pump suddenly fail to discharge its full capacity, this trouble is due to the fact that the water in the suction well has receded lower than the suction lift or the end of the suction pipe has become exposed. If the suction pipe is relatively small, thus having high velocity in it, its end need not be entirely uncovered, because air can be drawn down through the water by the whirlpools formed, and for this reason the bottom of the suction pipe should be at all times at least 4 diam. below the surface of the water. A leaky suction pipe must be immediately investigated and made tight. If a centrifugal pump discharges a fair quantity at its nozzle but fails to deliver the rated quantity under rated head, it is probable that the rotative speed is too low; the name plate should be consulted to make sure that the requirements called for are being fulfilled. Pumps should be opened from time to time to make sure that no foreign matter is lodged in the impeller or passages and no undue corrosion is taking place.

Automatic Starting Accessories. A centrifugal pump requires for tank or house service some device to start and stop it as the water level is lowered or raised to certain limits, and for this purpose automatic starters have been devised. A non-arcing float switch is designed to be used with automatically starting and stopping motor-driven pumps; it is not intended to carry the current delivered to the motor, but only to control a relay-magnet or solenoid, which operates a switch controlling the motor circuit. Range between high and low water may be varied by adjusting the distance between the knots in the cord which operates the float switch. There is no necessity of carrying large wires from the float switch to the motor; an ordinary lamp cord is sufficient between the float switch and the self starter; the float switch may then be installed near the tank and connected from any distance by means of lamp cord, or equivalent.

Pressure Regulator (gage type) is designed for use with the systems of automatically stopping and starting motor-driven pumps, working in connection with air or water systems, in which the governing is dependent on the pressure therein. The pressure regulator controls only the winding of a solenoid switch or relay and is a Bourdon gage with a special pointer provided with a non-oxidizable contact tip, which travels between two adjustable contact riders; the inner rider marks the low-pressure limit, and the outer rider the high-pressure limit. Circuit arrangements are such that but one side is connected to the gage at any time, so that danger of a short circuit is eliminated, also the possibility of trouble from vibration. Range of adjustment is limited only by the range of the gage itself, and the contact riders may be set for start-

ing and stopping the motor upon the variation of from 3 to 5 per cent. of the maximum pressure for which the gage is designed.

References. Great varieties of pumping units are offered by a large number of makers; many of them are quite satisfactory for usual requirements. Much additional useful information may be had from the publications of the best manufacturers. Reference is also made to "Centrifugal Pumping Machinery" by C. G. De Laval (McGraw-Hill Book Co., Inc., 1912); "Centrifugal Pumps," Loewenstein and Crissey (Van Nostrand, 1911); "Centrifugal Pumps," R. L. Daugherty (McGraw-Hill Book Co., Inc., 1915)

MISCELLANEOUS TYPES OF PUMPS

Humphrey Pump is a direct-acting gas explosion device, invented in England, improved in U. S., in which the expanding gas forces the water through the discharge pipe. Made by Humphrey Gas Pump Co., Syracuse, N. Y. Very few in use.—(See E. N., Dec. 2, 1909, Apr. 17 and Dec. 25, 1913, and Jan. 28, 1915.)

The d'Auria Pump has as a compensating device an oscillating column of water actuated by a plunger inserted between steam and water cylinders; it responds promptly, automatically and to the degree necessary to prevent sudden or material variation of speed due to change of load. D'Auria pump at Atlantic City was built by Henry G. Morris, Philadelphia, after designs by

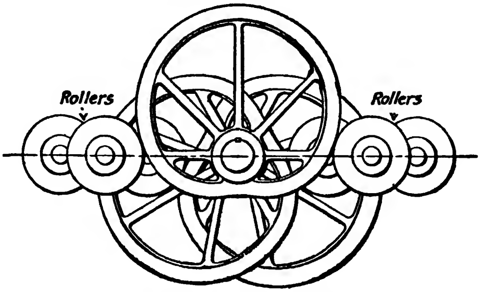


FIG. 289.—Triplex cam arrangement, Luitwieler engines.

inventor, Luigi d'Auria, and was placed in service Jan. 1, 1902. Jan. 29, 1903, a 10-hr. test developed 73,679,000 ft.-lbs. per 1000 lbs. of commercially dry steam, capacity being 3,411,000 gals. per day, with steam at 88.6 lbs. and total lift, including suction, 119.06 ft.; efficiency of steam end, 85.4 per cent.; of water end 93.6 per cent.; two combined, 80 per cent. Soon after pump was put into commission a blank flange on 20-in. force-main blew off, but adjustment was prompt, easy and effective, with no damage to the pump.—(Kenneth Allen, T. A. S. C. E., Vol. 74, 1911.)

These pumps have now only historical interest.

Riedler Pumps are high-speed reciprocating steam-driven pumps (50 to 60 r.p.m.) so-called because equipped with special all-metal grid-iron water valves actuated directly and independently from the engine. These valves were patented about 1890 by Prof. Alois Riedler, in Germany; they have been

used in a few important pumps in this country, notably one 20-mgd. unit in Chestnut Hill pumping station, Boston.

Non-pulsating Pumping Engine. Fig. 290 shows arrangement of two cams with top and bottom rollers in Luitwieler vertical pumping engines. Shape of cams is such that water is raised vertically at constant speed. One cam is shown in position of extreme upward travel, other has moved its rollers up incline, showing that there is no pause of water, because there are no dead centers. The makers claim such perfection in balance and water load as to produce no vibration when working under full load.

Diagrams, Fig. 291, show rate of suction and discharge during one revolution of six types of pumps: (1) Single-acting crank pump; (2) double-acting crank pump; (3) single-acting triplex pump, cranks set 120° apart; (4) double-acting crank triplex pump, cranks set 120° apart (triplex pumps give as high as 80 per cent. duty, Weston); (5) Luitwieler triplex cam pump, cams set 120° apart (Fig. 289); (6) submerged working barrel operated by two cams, single-acting, constant discharge.

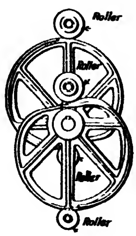


Fig. 290.

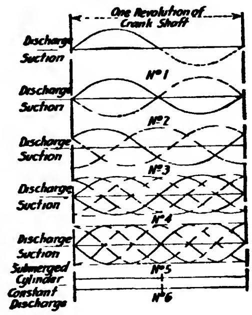


Fig. 291.

Electrical Pumping. For reasonable economy a motor must run at relatively high speed, so pumps adapted thereto must be chosen. Reciprocating power pumps require least power. Triplex pumps, either single- or double-acting, have advantage of reasonable uniformity of load; they are generally operated at 30 r.p.m. or less and require belting or other speed-reducing connections to motors. Mechanical efficiency of pump = 65 to 75 per cent.; of good belt transmission, 95 to 97 per cent.; of spur gearing, in good condition, 87 to 93 per cent. The greater number of installations have centrifugal pumps. Rotative speeds up to 1500 r.p.m., continuity of water flow, absence of shock and noise, and economy of space, make centrifugal pumps particularly adaptable to motor-drives.—(C. B. Burdick, in Proc. A. W. W. A., 1910.)

Hot-air Pumps. The "Reeco" Rider* and "Reeco" Ericsson hot-air pumping engines may be used in connection with pressure-tank systems as well as with elevated storage tanks. For shallow wells, where the pump is attached to the cylinder of the engine, and for domestic use where the daily consumption is small, and pressures from 25 to 75 lbs., the Ericsson type is recommended. The installation is simple and no air compressor is required; air and water are

* Notes from Catalog of Rider-Ericsson Engine Co., 20 Murray St., N. Y. City.

pumped at the same time by opening a pet-cock in the pump, and the engine is as easily managed with the pressure tank as with an elevated tank. Where deep-well pumps are used, a small air pump of the simplest construction is attached, with which there is no difficulty in getting the desired pressure.

The "Reeco" Rider engine has a greater capacity and is good especially for deep wells and for discharging to considerable hight or distance. Only a few minutes are needed to build a fire and when water has been pumped for a day's supply, the fire need not be replenished. An ordinary domestic can operate these engines. All parts can be examined and cleaned without difficulty and the fire can be replenished without stopping the engine; it can be made automatic by a kerosene-burning attachment. The operation is as follows: The compression piston first compresses the cold air in the lower part of the compression cylinder to one-third normal volume, when, by the upward motion of the power piston, and the completion of the downstroke of the compression piston, air is transferred from the compression cylinder through the regenerator into the heater without appreciable change of volume. The increase of pressure corresponds to the increase of temperature, and this impels the power piston up to the end of its stroke. The pressure remaining in the power cylinder, and reacting on the compression piston, forces the latter upward until it reaches nearly to the top of the stroke, when, by the cooling of the charge of air, the pressure falls to the minimum, the power piston descends and compression again begins. The heated air, in passing through the regenerator, has left the greater portion of its heat in the regenerator plates, to be picked up and utilized on the return of the air toward the heater. Generally the engine is placed near the source of water supply, and furnished with a rolling-valve pump bolted to the cooler. Where the suction is heavy, or there is a long line of horizontal suction pipe, use a special flap-valve pump. When the vertical distance from surface of the water to the surface of the ground is over 20 ft., use a special artesian-well pump, placed near surface of water or submerged, as more desirable. Connect to the engine by a pipe terminating in a stuffing box at top, which is bolted to the cooler, all water passing through the cooler. Through this pipe and stuffing box the pump rod passes. The depth to which pumps may be lowered depends on size of engine; with 6-in. engine they may be lowered 100 ft.; with 8-in., 200 ft.; and with 10-in., 400 ft. Pumps go into wells 2 in. diam. and upward.

"Reeco" Ericsson is a single-cylinder engine in which are two pistons—an air piston which receives and transmits power, and a transfer piston which transfers air at proper times from one end of cylinder to other. The operation is as follows: After lower end of cylinder has been sufficiently heated, which takes only few minutes, engine is started by hand by giving the wheel one or two revolutions. Air is first compressed in cold part of cylinder, then transferred to lower end, where it is instantly heated and expanded, thus furnishing power. Momentum of fly-wheel continues revolution. Same air is used continuously. Furnaces are arranged for burning any kind of fuel. When the water is farther from the surface than 20 ft., it becomes necessary to use a deep-well pump attachment; the engine must stand close to the well and in such a position that the pump rod will go directly from the engine to the pump. The pump should

be lowered into the water, so as to avoid work on the suction side. The deep well arrangement is to be used only on the 6-, 8- and 10-in. engines. Sizes refer to air cylinder. The pipe which connects the pump with the engine must be sufficiently large for the pump rod to work inside it, and also to allow sufficient space for the water to pass upward. Cut this discharge pipe into short lengths—8 to 10 ft.—as they are much more easily handled. The pump rod consists of pipe or wooden rods screwed together as they are lowered. The extreme depths to which pumps may be lowered are: 30 ft., for 6-in. engine; 60 ft. for 8-in. engine; 135 ft. for 10-in. engine.

OTHER ENGINES FOR RAISING WATER

Hydraulic Ram. *Use and Advantages.* Invented by J. Whitehurst, 1772; few improvements were made up to 1895. Most efficient pumping machine

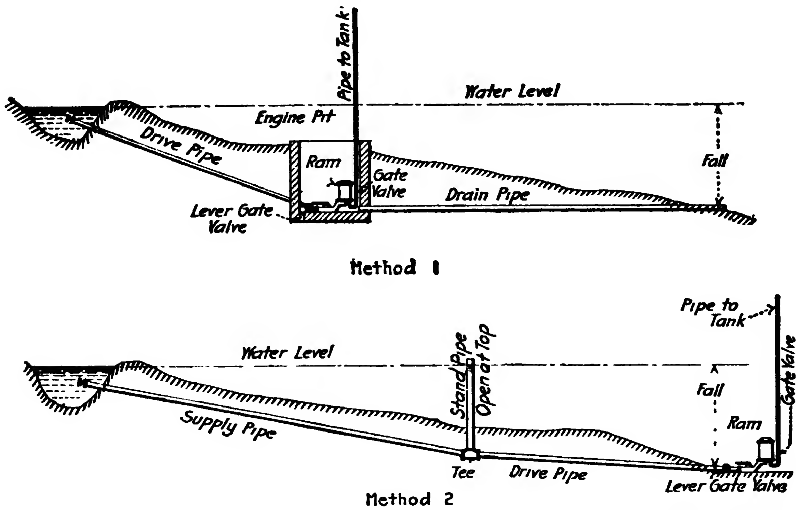


FIG. 292.—Two methods of securing fall for hydraulic rams. (See p 500.)

known; when well designed and operated under favorable conditions, 90 per cent. efficiency can be realized. It has combined functions of motor and pump, and must be so considered in comparisons of efficiency. In 1895 D. W. Mead designed for West Dundee, Ill., a ram with 43-ft. drive head and 2200 ft. of 10-in. cast-iron drive pipe, which is still operating successfully. S. B. Hill designed a waste valve which eliminates excessive hammering on seats, so that rams can act under higher heads. It is a balanced valve with downward discharge having 1-in. maximum movement. A Sterling 10-in. ram was tested at University of Washington; valve opening was $\frac{3}{4}$ in.; strokes per min. ranged from 44 to 97; quantity pumped, 0.1 to 0.8 c.f.p.s.; additional water used for power ("wasted"), 0.8 to 1.3 c.f.p.s.; supply head, 50 ft.; pumping head, 61 to 278 ft.; delivery head, 111 to 327.5 ft.; average efficiency, by D'Aubisson's rule, 89 per cent.; by Rankine's, 85 per cent. No difficulty should be encountered in operating under supply heads of 1 to 100 ft. and

delivery heads of 5 to 500 ft.—(S. B. Hill, Proc. Pacific Northwest Soc. Eng'rs., Nov., 1911.)

Rife Automatic Hydraulic Ram will pump water 500 ft. high, 30 ft. high* for every foot of fall; minimum requirements, 3 gals. per min. under 3 ft. head. Has large air chamber, positively fed at each stroke, maintaining ample air cushion. Pumping capacities up to 1,000,000 gal. per day. Double-acting rams pump pure water by power of impure water without mixing. Can be installed with pneumatic tanks where overhead tanks are objectionable. By this means water at low head may be used for raising a portion of same or other water to higher level than supply. If a ram be placed 2 ft. or more below the surface of the water in a supply and power water that escapes be drained away, a constant flow will be delivered to the higher point. It is necessary to replace valves once in about 2 yrs.

A Double-acting, or Double-supply, Type of Ram is shown in Fig. 293. Without regard to double-supply feature, suppose no opening at H, valve *B* open; water from source flows down drive-pipe *A* and escapes through *B*

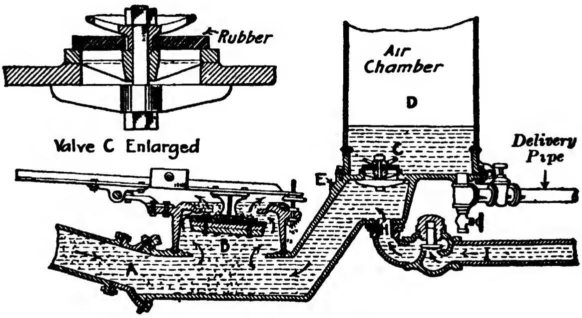


FIG. 293.—Cross-sectional elevation of hydraulic ram.
(E. N., Dec. 31, 1896.)

until pressure due to increasing velocity is sufficient to close *B*. At moment flow through *B* ceases, inertia of moving water produces ramming stroke, which opens valve *C*, and compresses air in chamber *D* until pressure of air plus pressure due to head of water in main is sufficient to overcome inertia of moving water in drive-pipe. At this instant water in drive-pipe has come to rest, and air pressure being greater than static head alone, direction of motion is reversed and *C* closed. Water in drive-pipe is then moving backward, and with closing of *C* tendency to vacuum is produced at base of drive-pipe; this negative pressure causes *B* to open again, completing cycle of operations. At moment of negative pressure, small snifting valve *E* admits a small quantity of air, and at following stroke this passes into chamber, which would otherwise gradually fill with water. Weight of waste-valve *B* must be greater than pressure of statical head of water on its under side so that it may open when column of water comes to rest. In the machine described valve *B* is made as light as consistent with necessary strength; negative pressure at end of stroke is relied upon to open the valve. Waste mechanism of Rife ram consists of a large port with flat, ample opening and a large rubber valve with

* Within limits of ratio of fall to lift giving reasonable economy. See footnote, p. 502.

counterweight and spring seating, removing almost all jar at closing. Valve C in air chamber consists of a rubber disk with gridiron ports and convex seats, fastened at center and lifting at circumference. Efficiency of 82 per cent. is claimed. A most important detail in which Rife rams differ from ordinary hydraulic rams is the waste valve; a counterweight on a projecting arm permits adjustment to suit varying heads and lengths of drive-pipe. By adjusting the counterweight so that the valve is nearly balanced, the valve comes to its seat quickly after the flow past it begins. The result is that the ram makes a great number of short, quick strokes which are much easier on mechanism than slower and heavier strokes.

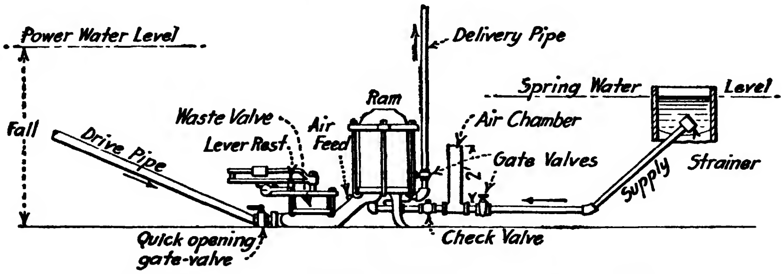


FIG. 294.—Arrangement of various accessories of ram.

Efficiency. D'Aubisson's formula for calculating efficiency is

$$E = \frac{Q_s(H + H_s)}{QH}$$

Q is c.f.p.s. *flowing* in drive-pipe; Q_s is c.f.p.s. flowing to standpipe through discharge* pipe; H is hight (in ft.) from escape† valve to level of reservoir which feeds drive-pipe; H_s difference in level (in ft.) of water in supply reservoir and water in standpipe or other receiving vessel. Rankine formula for efficiency is

$$E = \frac{Q_sH_s}{(Q - Q_s)H}$$

D'Aubisson's is correct, considering mechanism as receiving energy at one end and delivering it at other, while if machine is considered as elevating water only from one reservoir to other, Rankine's is correct. When a pipe is attached at H (Fig. 293), engine is termed double-acting; water which is purer than the water used to drive may then be supplied through I, and by proper adjustment of relative flow of impure driving water, and pure supply, the ram may be made to deliver only pure water. This method is used where the supply of pure water is limited.

Length of Drive-pipe. To insure sufficient air being fed automatically at each stroke, it is imperative that the drive-pipe should be of proper length, which is determined by the fall to the ram and the hight to which water is delivered. In order to obtain desired fall, it is frequently necessary to convey water a greater distance than the length of the drive-pipe used.

Directions for Setting Rams. A ram should be on a level, firm foundation; it need not be fastened. Drive-pipe must be laid on a perfectly straight incline,

* Same as delivery pipe. † Same as waste valve.

without bends or curves except where the pipe enters the ram, and this should be made by bending the pipe; don't use fittings. Upper end of drive-pipe should be sufficiently below surface of water, so that it cannot suck air—1 ft. or more. All drive-pipes should have good open strainers on upper ends. Be sure the drive-pipe is air-tight. Use a quick-opening gate valve laid on its side. The delivery pipe can be laid with necessary bends, according to usual practice in water pipes. Connect all pipes before starting the ram and leave them uncovered until it has been shown there are no leaks. Use screw-gate valves on delivery pipes. Start the ram by forcing the outside escape valve open by hand and allowing it to close; repeat this operation until enough water has been forced into the delivery pipe to create back pressure and cause the ram to work automatically. At the start put the slide weight over the center of the balance rod and fasten securely. By raising and lowering the leather covered rest under the balance valve lever, by means of two nuts thereon, the opening of the valve can be lessened or increased, which will lessen or increase the quantity of water used and delivered. The greater the

Table 148. Data on Hydraulic Rams (Rife)

Number	Dimensions						Size of drive-pipe, in.	Size of delivery pipe, in.	Gals. per min required to operate		Least fall recommended, ft	Weight, lbs.
	Hight, ft.	in.	Length, ft.	in.	Width, ft.	in.			Min.	Max.		
10	2	2	2	10	1	0	1¼	¾	2½	6	3	150
15	2	2	3		1	0	1½	¾	6	12	3	175
20	2	5	3	3	1	2	2	1	8	18	3	225
25	2	5	3	4	1	3	2½	1	12	28	3	250
30	2	7	3	7	1	3	3	1¼	20	40	3	275
40	3	7	4	9	1	8	4	2	30	75	4	600
60	4	8	6	0	2	3	6	3	75	150	5	1200
80	6	4	7	0	3	0	8	4	150	300	5	2200
120*	8	9	8	4	2	8	12	5	375	700	6	3000

* See example in Table 149 . By using "Batteries" unlimited quantity can be delivered.

Table 149. Delivery of Hydraulic Rams, Gals. Per Day

Power head, ft	Pumping head, ft.																
	4	10	15	20	30	40	*50	60	70	80	90	100	120	140	160	180	200
2	540	192	128	96	64	43	29	24	..		. .			.			
3	.	301	192	144	96	72	58	43	37	27	24	...	.				
4	.	432	256	192	128	96	77	64	55	43	38	29	24	...			
5	.	540	345	240	160	120	96	80	69	60	53	43	30	26			
6		...	432	302	192	144	115	96	82	72	64	57	43	31	27	24	...
7		...	505	378	235	168	134	112	96	84	75	67	50	36	31	28	25
8		.	....	432	270	192	154	128	110	96	86	77	64	55	43	38	29
9			. .	485	300	216	173	144	124	08	96	86	72	62	54	43	39
*10			. .	540	360	252	*192	160	137	120	107	96	80	68	60	53	43
12				.	430	301	230	192	165	144	128	115	96	82	72	64	57
14					505	353	270	224	192	168	150	135	112	96	84	75	67
16					...	432	323	257	220	192	171	154	128	110	96	85	77
18					..	486	390	303	247	216	192	173	144	124	108	96	86
20			.			540	430	336	288	240	214	192	160	137	120	107	96
22							475	370	303	264	235	212	176	151	132	118	105
24							520	405	346	288	256	230	192	164	144	128	115
26							....	470	375	328	278	250	208	178	156	139	125
28							....	505	430	354	300	269	224	192	168	149	134
30							. .	540	465	405	336	288	240	206	180	160	144

* *EXAMPLE*
With supply of 700 gals. per min , 10 ft. fall, 50 ft. elevation, No. 120 engine will deliver 134,400 gals. per day:
700 × 192 = 134,400

Multiply factor opposite "power head" and under "pumping head" by number of gals. per min. *used* by engine and result will be number of gallons *delivered* per day. Efficiency is governed by ratio of fall to pumping head; 75 per cent. for ratio of 1 to 2½; 70 per cent. for ratio of 1 to 3; 66⅔ per cent for ratio up to 1 to 18; 60 per cent. for ratio up to 1 to 23; 50 per cent. for ratio up to 1 to 30. (Rife Hydraulic Engine Co., 111 Broadway, New York City.)

number of strokes, the *less* water is used and pumped; the fewer strokes, the *more* water used and pumped, per stroke. In each installation a ram runs best at a certain opening of the waste valve, to be determined only by experiment. By changing the nuts on the lever rest, a position will be found where the ram runs with an easy, even stroke; then it is in proper working order. Air feed should be sufficiently open at all times to allow a small spray of water to escape at each stroke. Length of drive-pipe is most important and is governed by ratio of fall to elevation. If too long or too short, automatic supply of air is interfered with and efficiency impaired.

Windmills. Horsepower of windmills of best construction is proportional to squares of diam. and inversely as velocities; for example, a 10-ft. mill in a 16-mi. breeze will develop 0.15 hp. at 65 r.p.m.; and with same breeze: 20-ft. mill, 40 rev., 1 hp.; 25-ft. mill, 35 rev., 1¾ hp.; 30-ft. mill, 28 rev., 3½ hp.; 40-ft. mill, 22 rev., 7½ hp.; 50-ft. mill, 18 rev., 12 hp.

Increase in power from increased velocity of wind is equal to square of its proportional velocity; for example, the 25-ft. mill rated above for a 16-mi. wind will, with a 32-mi. wind, produce 4 × 1¾ = 7 hp. A windmill will run and produce work in a 4-mi. breeze. (Table of wind velocities on p. 627.)

Table 150. Capacities of Windmills, Gals. per Minute*

Designation of mill	Velocity of wind in miles per hr.	Revolutions of wheel per min.	Elevation, ft., raised						Equivalent actual useful hp. developed
			25 ft.	50 ft.	75 ft.	100 ft.	150 ft.	200 ft.	
8½	16	70–75	6.192	3.016	.....	......	......	....	0 04
10	16	60–65	19.179	9.563	6.638	4.750	......	....	0 12
12	16	55–60	33.941	17.952	11.851	8.435	5.680	....	0.21
14	16	50–55	45 139	22.569	15.304	11.246	7.807	4.998	0.28
16	16	45–50	64 600	31.654	19.542	16.150	9.771	8 075	0.41
18	16	40–45	97 682	52 165	32.513	24 421	17.485	12 211	0.61
20	16	35–40	124 950	63.750	40 800	31.248	19 284	15 938	0.78
25	16	30–35	212.381	106.964	71.604	49.725	37.349	26 741	1.34

STEAM BOILERS

Boiler Horsepower, according to standard adopted by Am. Soc. M. E., is 30 lbs. water evaporated per hr. at a gage pressure of 70 lbs. per sq. in., and from a temperature of 100° F. In calculating the horsepower of steam boilers, allow (*a*) tubular boilers, 15 sq. ft. of heating surface to 1 hp.; (*b*) flue boilers, 12 sq. ft.; (*c*) cylinder boilers, 10 sq. ft. Well-designed boilers, under successful operation, will evaporate from 7 to 10 lbs. of water per lb. of first-class coal. Each square foot of heating surface is considered sufficient to evaporate 2 lbs. of water; therefore for an engine using 30 lbs. of water per horsepower per hour, each horsepower of the engine requires 15 sq. ft. of heating surface in the boiler. To evaporate 1 cu. ft. of water requires the consumption of 7.5 lbs. of ordinary coal, or about 1 lb. of coal to 1 gal. of water. Based on 34.5 lbs. of water per horsepower per hour, boiler horsepower × 0.069 = gals. per min.

Selecting Boilers. "Horsepower" is frequently used when estimating capacity of boilers, and buyers ask for boilers of a certain horsepower. This is an improper way to buy unless the heating surface per horsepower is clearly

* U. S Wind Engine & Pump Co

known. One bidder might offer a boiler with ample heating surface, another one with much less. Both boilers may develop the required horsepower, but the one with insufficient heating surface might do it only at increased cost for fuel. Boilers should not be bought with less than 1 sq. ft. of heating surface for every 3 lbs. of water to be evaporated; in other words, with not less than 10 sq. ft. of heating surface per b.hp. It will usually be found good practice and economical to provide reserve power in the boilers and ample heating surface, particularly if the boiler plant is designed on the test duty of the engine, which is never realized in actual practice. Best efficiency under ordinary working conditions is usually obtained by allowing very nearly 12 sq. ft. of heating surface per b.hp.

Division of Heating Surface into Units. First determine accurately the maximum number of pounds of steam that will be used. The maximum evaporation of a boiler is limited mainly by the quantity of coal which can be burned upon the grate. By dividing the total number of pounds of steam to be evaporated per hour by 3, the total heating surface may be obtained with fair accuracy. The next step is the subdivision of this heating surface into a proper number of boilers. To evaporate a given quantity of water into steam, it is necessary to generate a certain amount of heat by combustion of fuel. The controlling factors are: kind of coal, area of grate surface, and draft. Ample grate surface is desirable. The kind of coal to be used should be determined before designing the boiler plant, the cost of various fuels available and their calorific value, or relative evaporative power. With good coal, low in ash, approximately equal results may be obtained with large grate surface and light draft or with small grate and strong draft. Bituminous coal, low in ash, gives best results with high rates of combustion, provided ratio of grate surface to heating surface is properly proportioned. Coals high in ash require a comparatively large grate surface, particularly if the ash is easily fusible, tending to choke the grate. Where a strong draft is available a smaller grate may be used than with moderate draft, as a thicker bed of fuel can be carried. When it is intended to burn low grades of fuel, provision for a large grate ought to be made in the first place. Then, if it is later desired to change to a fuel of a better grade, this can be done by reducing the size of the grate by bricking off a portion of it. If certain that there never will be a desire to use a poorer fuel, it would be uneconomical to provide larger grates than necessary.

Table 151 gives a very approximate estimate of the relative evaporative power of several kinds of coal, or the water that 1 lb. of coal will evaporate, the pounds of coal that may be economically burned per sq. ft. per hr., and the ratio of grate surface to heating surface with ordinary drafts.

Table 151. Evaporative Power of Coals

	Pounds of water 1 lb. of coal will evaporate, with steam at 212° F.	Pounds of coal per sq. ft. of grate, per hr.	Ratio of heating to grate surface
Best bituminous	9.0	21	55
Ordinary bituminous	8.0	20	46
Anthracite (nut or larger)	8.0	13	32
Anthracite (buckwheat or rice)	7.0	12	26

See also p. 461.

Type of Boiler. Three types are in general use, but there are many variations in these types: (1) plain return tubular boiler set in brick-work; (2) firebox boiler, or internally fired boiler of marine type, which requires no brick setting and is complete when it leaves the boiler shop; (3) water-tube boiler, set in brick-work, with hot gases passing around the tubes, of which the Babcock & Wilcox is now the oldest and best known type. A chief feature in the selection of the type of boiler is the steam pressure to be carried. Return tubular boilers are made up to 84 in. diam. and 20 ft. long to carry 150 lbs. pressure per sq. in., but boilers of this type should rarely, if ever, be used for pressures exceeding 130 lbs. Initial pressure for vertical, triple-expansion engines frequently runs from 150 to 190 lbs., requiring another type of boiler. If initial steam pressure is reduced, purchaser has to pay interest on increased size of cylinders of engine. Return tubular boilers are seldom made in sizes over 200 hp., and hence are not to be considered for large units. For high pressures, 125 lbs. or more, water tube, or some form of internally fired boiler, in which the shell plates are not exposed to the high temperature of the furnace, is safer than the horizontal tubular boiler, because the shell plates and seams of the latter must be thick in large units, and, being exposed to the hottest part of the fire, are likely to give trouble, especially if the water contains scale or sediment-forming elements. The internally fired type is economical of space, saves repairs to brick settings and has good steaming qualities, but is not cheap. Internally fired boilers are more expensive than externally fired, though the cost of setting and foundations of the latter may bring the total cost to practically the same figure. Internally fired boilers, with corrugated fire boxes, of Fox or Morrison type, excel for high pressure. Water tube boilers have a great advantage in that high pressures may be carried with perfect safety; they are usually employed in large central stations with high pressure and units of 300 to 500 hp. When proportions are suitable, and conditions favorable, the different types of boilers give substantially the same efficiency. Water tube boilers seldom, if ever, explode, although tubes sometimes burst; contain less water within the shell, and consequently, steam can be raised quickly. On the other hand, this type is flashy and it is harder to maintain a uniform steam line than with horizontal tubular boilers, or the internally fired marine type. Relative merits of boilers should be considered with reference to: durability, accessibility for repairs, facility for cleaning and inspection, space requirements, adaptability to the type of furnace and stoker desired, capacity, and cost of boiler and setting.

Superheat.—Installation of a superheater is equivalent to an increase in boiler capacity. Superheaters may be independently fired, or may be connected with the boiler. Engineers are not agreed as to which arrangement gives the more economical returns. Requirements for a successful superheater are: security in operation, or minimum danger of overheating; economical use of heat applied; no exposure of joints to fire; provision for free expansion; disposition such that joints can be cut out or repaired without interfering with operation of plant; ease of application to existing plant. Independently fired superheaters have the following advantages: degree of superheat may be varied, independently of performance of boiler; can be placed at any desirable

point; repairs can readily be made without shutting down the boilers. Some disadvantages are: separate firing and extra attention; extra piping; extra space; greater initial cost. Standard practice in this country tends toward the superheater contained within the boiler setting. In pumping stations 100° F. of superheat is sufficient. This corresponds to an increase of 8 to 10 per cent. in horsepower of boiler, insures practically dry steam at cut-off, and, therefore, produces a material reduction in losses due to cylinder condensation. Higher temperatures interfere with lubrication, and sometimes cause warping of valves. With 100° of superheat, and a steam pressure of 125 lb., producing a temperature of 425° F., no difficulties are ordinarily met. If superheat is used, metallic packing gives best results for piston rods and valve stems. If highly superheated steam is to be used, valves should be especially designed. If superheat is to be used, cast iron should be eliminated, as far as possible, from the steam piping system, flanges should be wrought steel, fittings should be cast steel, and valves should be carefully selected.

Superheaters are especially effective in small waterworks pumping stations with low-duty, triple-expansion engines. Cost of superheater installation amounts to but $8 or $10 the horsepower, including added cost of piping. Saving in fuel ranges from 10 to 20 per cent. Cost of repairs and maintenance is low, and if properly taken care of, they give little trouble.

Table 152. Percentage of Fuel Saved by Heating Feed Water*

Steam pressure 60 lbs. gage

Temperature of water entering heater, ° F.	Temperature of water entering boiler, degrees Fahrenheit											
	120	140	160	180	200	202	204	206	208	210	212	214
32	7 5	9 2	10 9	12 4	14.3	14 5	14 6	14 8	15.0	15 1	15 3	15 5
40	6 8	8 6	10 3	12.0	13.7	13.9	14 0	14.2	14 4	14.6	14 7	14.9
50	6 0	7 8	9 5	11.2	13 0	13.1	13 3	13 5	13.7	13 8	14 0	14 2
60	5 2	7.0	8 7	10.5	12.2	12 4	12.5	12.7	12 9	13 1	13 2	13 4
70	4 4	6 2	7 9	9.7	11 4	11 6	11.8	12 0	12 1	12 3	12 5	12 7
80	3 4	5 3	7 1	8 9	10.6	10 8	11.0	11.2	11 4	11 5	11 7	11.9
90	2 7	4.5	6 3	8 1	9 8	10 0	10.2	10.4	10 5	10 7	10 9	11 1
100	2 0	3.6	5 4	7 2	9.0	9 2	9 4	9.6	9 7	9 9	10 1	10 3
110	0.9	2.7	4 5	6 4	8 2	8 4	8 6	8 7	8 9	9 1	9 3	9 5
120	...	1 8	4 0	5.5	7 3	7 5	7 8	7 9	8 1	8.3	8.4	8 6

Feed-water Heating. Exhaust-steam feed-water heaters are used to heat water fed to boilers with steam exhausted by pumping engines and auxiliaries, and the saving is so great that it is strange that small pumping stations are so frequently constructed without them. Generally speaking, for every 11° F. that feed water is warmed, there is a saving of 1 per cent. in fuel. With sufficient exhaust steam available, feed water at 50° to 60° can be raised to practically 200°, thus saving 12 per cent. of fuel. In pumping stations it is usually economical to use condensing apparatus in connection with the main pumps. If there is no station lighting plant, this leaves the exhaust steam from the auxiliaries only for feed-water heating, and, if feed water supply is cold, the heat contained in the exhaust from the auxiliaries is rarely sufficient to raise the feed water to more than 100° to 125° F. It is well to carry all condensation from jackets on main engines as well as from steam piping to a hot well. This

* National Pipe Bending Co., New Haven, Conn.

will usually raise feed water with initial temperature of 60° to at least 100°, where steam pipe and jackets are effectively covered, and if it passes thence through the feed-water heater, the resulting temperature will be about 150°. One combination that works out favorably under conditions outlined, is to return the condensation to a hot well, carry the exhaust steam from independent auxiliaries to the feed-water heater and extract sufficient heat from the flue gases, by means of an economizer, to return the water to the boilers at a temperature of from 200° to 225° F. Saving in coal effected is approximately 15 per cent., with initial feed water temperature 60° F. Complete outfit—hot well, piping, feed-water heater and economizer—should not cost in excess of $8 to $10 per hp. Provide thermometer wells and thermometers for obtaining temperatures resulting from various parts of the feed-water system.

Fuel Economizers.* Factors to be considered before installing are: nature of auxiliaries and heat to be derived from them; methods of heating feed water, whether atmospheric heaters are used, and whether all, or part, of exhaust steam is used for heating; initial temperature of feed water and whether feed water is taken from hot well or from a cold supply; probable rise in temperature due to installation of economizer; cost of economizer; cost of additional building space; reduction of boiler heating surface made possible by economizer; extra cost of stack, or forced draft apparatus necessary to compensate for loss of draft due to economizer; interest, depreciation, maintenance and operating cost, and insurance on all of those elements which result from installing economizer.

Steam Piping. The best pipe and fittings will prove most economical. Since steam is hotter than air, it must lose heat in its passage through the pipes. Pipes, therefore, should be of minimum size consistent with not too great pressure losses from increased friction. The piping should take the most direct course from boiler to engine, and boiler and engine should be as near together as possible. Steam piping should never be erected until an assembly drawing of the entire installation, giving locations of all valves and fittings, has been prepared, in order to avoid interferences and unnecessary bends and fittings. Isometric or perspective sketches usually are clearer than the conventional plan and elevation. Steam pipes, feed-water piping, boiler steam drums, receivers, separators, etc., should be covered with heat insulating material to reduce radiating losses to a minimum. Loss of heat from a bare steam pipe may be taken at about 3 B.t.u. per sq. ft. per hr. per deg. difference in temperature. By properly applying any good commercial covering, from 75 to 90 per cent. of heat loss by radiation may be prevented. A great difficulty in steam piping is provision for expansion and contraction due to change in temperature, and this difficulty increases with superheated and high pressure steam. Pipes may never be immovably fixed. They should be carried on rollers and supports which allow free contraction and expansion. Long radius bends and double swing expansion joints will materially aid in preventing breakage and leaks. Between 60° F., and temperature of steam at 200 lbs. pressure, the expansion in a pipe will be over 4 in. in 100 ft. A good, practical rule is to allow for 1 in. expansion in every 50 ft. of pipe. With superheated

* Fuel economizer is a long, narrow brick chamber between boilers and stack, or chimney, through which flue gases pass, containing many vertical cast-iron tubes in which feed water circulates, entering at opposite end of chamber from gases.

steam, expansion will be greater. (Paragraphs "Selecting Boilers" to "Steam Piping," by N. S. Hill, Jr., Proc., Municipal Engrs. of N. Y., 1911.)

Rules for Management and Care of Boilers Adopted by Hartford Steam Boiler Inspection and Insurance Co., 1870.* (1) *Condition of Water.* First duty of engineer when he enters boiler room in morning is to ascertain how many gages of water there are in boilers. Never unbank or replenish fires until this is done. (2) *Low Water.* Immediately cover fire with ashes; or if no ashes are at hand, fresh coal. Don't turn on feed under any circumstances, nor tamper with nor open safety valve. Let steam outlets remain as they are. (3) *Foaming.* Close throttle, and keep closed long enough to show true level of water. If that level is sufficiently high, feeding and blowing will usually suffice to correct evil. In case of violent foaming, caused by dirty water or change from salt to fresh, or *vice versa*, in addition to action above stated, check draught and cover fires with fresh coal. (4) *Leaks.* When discovered they should be repaired as soon as possible. (5) *Blowing off.* Blow down under pressure not exceeding 10 lbs. Where surface blow-cocks are used, they should be opened wide once a day—oftener if water contains much sediment. Time required to open wide and close valve is long enough. (6) *Filling Boiler.* After blowing down, allow boiler to become cool before filling again. Cold water pumped into hot boilers is very injurious from sudden contraction. (7) *Exterior of Boiler.* Care should be taken that no water comes in contact with exterior of boiler, either from leaky joints or other causes. (8) *Removing Deposit and Sediment.* In tubular boilers hand-holes should be often opened, and all collections removed from over fire. Also, when boilers are fed in front and blown off through same pipe, collection of mud or sediment in rear end should be often removed. (9) *Safety Valves.* Raise safety valves cautiously and frequently, as they are liable to become fast in their seats (ordinary weight and lever style) and useless for purpose intended. (10) *Safety Valves and Pressure Gage.* Should gage at any time indicate the limit of pressure allowed, see that safety valves are blowing off. (11) *Gage Cocks, Glass Gage.* Keep gage cocks clear and in constant use. Glass gages should not be relied on altogether. (12) *Blisters.* When blister appears there must be no delay in having it carefully examined, and trimmed or patched, as case may require. (13) *Clean Sheets.* Particular care should be taken to keep sheets and parts of boilers exposed to fire perfectly clean; also all tubes, flues, and connections well swept; particularly necessary where wood or soft coal is used. (14) *General Care of Boilers and Connections.* Under all circumstances keep gages, cocks, etc., clean and in good order, and things generally in and about engine and boiler room in neat condition.

Firing Boiler. Coal of depth up to 12 in. is more effective than less depth. Admission of air above grate increases evaporative effect, but diminishes the rapidity of it. Air admitted at bridge-wall effects a better result than when admitted at door, and when in small volumes, and in streams or currents, it arrests or prevents smoke. It may be admitted by an area of 4 sq. in. per sq. ft. of grate.

Combustion. The rate of combustion in a furnace is computed by the pounds of fuel consumed per sq. ft. of grate per hr. The size of coal must

* For more complete rules, revised, reader is referred to The Locomotive (published by Hartford Steam Boiler Inspection & Insurance Co.), July 25, 1911.

be reduced and the depth of fire increased directly, as the intensity of draught is increased. On 1 sq. ft. of grate can be burned an average from 10 to 12 lbs. of hard coal, or 18 to 20 lbs. soft coal, per hr. with natural draft. With forced draft nearly double these amounts can be burned.

Boiling Point of Water at mean atmospheric pressure at sea level is 212° F. At an absolute pressure of 6 lbs. per sq. in. (17.70 in. vacuum), this drops to 170.1° F.; while at 3 lbs. absolute (23.83 in. vacuum) the boiling point is only 141.6° F.

Steam. Steam arising from water at boiling point (212° F.) has a pressure equal to the atmosphere (14.7 lbs. at sea level). One cubic inch of water evaporated under ordinary atmospheric conditions is converted into 1 cu. ft. of steam. The specific gravity of steam at atmospheric pressure is 0.411 that of air at 34° F., and 0.0006 that of water at the same temperature; 27,222 cu. ft. of steam weigh 1 lb.; 13,817 cu. ft. of air weigh 1 lb. (both at atmospheric pressure).

Air Pumps and Condensers are required to pump cold water, hot water and air. If duplex movement be used, while one piston is forcing water, the piston on opposite side may be encountering air or vapor, and meeting no resistance; consequently that piston will race forward until it meets solid matter, resulting in throwing the steam valve on the other side too soon, thereby making the pump short-stroke. Where duplex air pumps and condensers are used, unless machine is so small that it must be pumping water steadily in order to provide enough to condense steam—and such are too small to be economical—it will be noticed that they do shorten their strokes, and thus impair their efficiency and economy. Single-acting air pumps and condensers are self-contained machines; each valve being thrown by the action of its own piston, it must complete its stroke in length whether the piston is moving in air, water or vapor; consequently nearer approach to theoretical quantity of water is obtained.

Jet condensers require much less water than surface condensers, but the water so used is wasted ordinarily. The speed of the pump and quantity of injection water can be regulated to suit the working conditions of the engine. It will quickly produce and maintain a high vacuum, removing nearly all the (atmospheric) back pressure from the piston of the engine. Being independent, it can be started and a vacuum formed before starting the engine. It will require less power than a connected air pump; the power being taken directly from the boiler, the engine is relieved of that much load. It requires no additional pump for the injection water; the air pump will lift the water from any point within the limit of suction.

Surface Condensers should have about 2 sq. ft. of tube (cooling) surface per horsepower for a compound steam engine. Ordinary engines will require more surface according to the economy in use of steam. It is absolutely necessary to place air pumps below the condensers to get satisfactory results.

In Ordering Condensers give: (1) Diameter of steam cylinder and stroke of engine. (2) Revolutions per minute. (3) Steam pressure in boiler. (4) Maximum point of cut-off, or has the engine a plain slide-valve? (5) Maximum temperature of the water, to be used for condensation, and is the water obtained from wells, stream, or pond? (6) Distances vertically and horizont-

ally from the surface of the water supply to the floor on which the condenser can be placed; (7) state whether water is fresh or salt, clean or dirty; (8) send indicator cards of the engine if possible.

Operating hight of condenser above injection supply should be as little as possible; 18 or 20 ft. ought not to be exceeded. For high lifts of injection water, a charging valve and overhead supply will be found very advantageous in starting the condenser; this supply should be cut off as soon as vacuum is had. Start the condensing apparatus in advance of the engine, and shut down the engine before shutting down the condenser. It is estimated that 20 volumes of water absorb 1 volume of air; hence, if means were not taken to remove this air from the condenser, it would fill it and the cylinder and destroy the vacuum.

The *effect* of a good condenser and air pump should be to make available about 10 lbs. more mean effective pressure, with the same terminal pressure; or to give the same mean effective pressure with a correspondingly less terminal pressure. When the load on the engine requires 20 lbs. mean effective pressure, the condenser does half the work; at 30 lbs., one-third the work; at 40 lbs., one-fourth, and so on. It is safe to assume that practically the condenser will save from one-fourth to one-third of the fuel, and can be applied to any engine, cut-off or throttling, where a sufficient supply of water is available.

Table 153. Relation of Vacuum to Temperature Fahr., of Feed Water

Vacuum, in.	Temp., Degrees	Vacuum, in.	Temp., Degrees
00	212	27½	112
11	190	28½	92
18	170	29	72
22½	150	29½	52
25*	135		

Condensing Engines Require from 20 to 30 gals. of water, at average low temperature, to condense steam represented by every gallon of water evaporated in boilers supplying engines—approximately for most engines from 1 to 1½ gals. condensing water per minute per indicated horsepower. With a limited supply water is used over and over, being cooled by circulation through a "cooling tower," or a system of spray nozzles and basin.

FUELS

Weights of Fuels (for approximate computations). Petroleum weighs 6.5 lbs. per U. S. gallon, 42 gals. to the barrel. One cubic foot of anthracite coal weighs about 53 lbs.; bituminous coal, 47 to 50 lbs.

Specifications for Coal. (Abridged from New York City "Standard Specifications for Furnishing, Delivering, Storing and Trimming Coal.") *Kinds* and *Sizes.* "Anthracite" shall mean coal mined in the anthracite districts of Pennsylvania. "Semibituminous" shall be accepted in its usual commercial meaning. Not less than 90 per cent. shall pass over the minimum screen, and of broken, egg, stove and chestnut, not less than 90 per cent. shall pass through the maximum screen.

* Usually considered the standard point of efficiency, condenser and air pump being well proportioned.

Table 154. Sizes and Standard Analyses of Anthracite Coal

Tests with stationary screen inclined 45°

Names of sizes of coal	Shall pass through square mesh screen of clear opening, in.	Shall pass over square mesh screen of clear opening, in.	Per cent. moisture as delivered	Per cent. ash, dry coal	Per cent. volatile combustible, dry coal	Per cent. volatile sulphur, dry coal	B.t.u. per lb, dry coal
Broken....	4¼	2¾	4	11	8	1 5	13,200
Egg.. . ..	3	2	4	11	8	1.5	13,200
Stove......	2½	1¼	4	12	8	1 5	13,000
Chestnut.	1½	¾	4	12	8	1 5	13,000
Pea........	¾	½	5	17	8	1.5	12,300
Buckwheat							
No. 1.	½	¼	6	18	8	1.5	12,200
No. 2. .	¼	3/16	6	19	8	1 5	12,100
No. 3. .	3/16	3/32	6	19	8	1 5	12,000

Table 155. Sizes and Standard Analyses of Semibituminous Coal

Names of sizes of coal	Through bars separated	Over bars separated	Per cent moisture as delivered	Per cent ash, dry coal	Per cent. volatile combustible, dry coal	Per cent. volatile sulphur, dry coal	B.t.u. per lb., dry coal
Run of mine.	Coal as it comes from mine, unscreened.	. .	3 0	10	25	1 75	13,800
Lump	...	1¼ in.	2 5	9	25	1 5	14,000
Nut	1¼ in.	¾ in.	2 5	9	25	1.5	14,000
Slack.	¾ in.		3.0	11	25	1 75	13,600

Payment. Coal of quality superior to standard analyses shall be paid for as coal conforming to the standard, except that weight shall be corrected for moisture in excess of the specified percentage.

Method of Sampling. From deliveries exceeding 25 tons, but not exceeding 100 tons, a sample of 200 lbs. shall be taken; for deliveries in excess of 100 tons, approximately one-tenth of 1 per cent. of the quantity delivered. The gross sample shall be broken by hand, or by crusher, to approximately pea size or smaller, and reduced by successive quarterings to not less than 5 lbs. for laboratory tests. Samples for the determination of moisture content will be taken at the point of weighing and immediately collected in moisture-tight receptacles.

Method of Analysis. A portion of the sample shall be analyzed upon its receipt at the laboratory. Moisture, ash, volatile sulphur and volatile combustible matter shall be determined by proximate analysis, and the heating value by an oxygen bomb calorimeter. The remainder shall be preserved for 30 days, in proper custody, after notification to the contractor of the results of the analysis, for a check analysis if required. The moisture determined in the first analysis shall be final. If the first analysis shows a deficiency in the heat units or an excess of ash, volatile combustible or volatile sulphur, and if the contractor questions the first analysis, he may, within 10 days after notification of the first analysis, request, in writing, a second

analysis. This second analysis shall be final. If the aggregate deductions in gross weight computed on the check analysis equals or exceeds the deductions computed on the first analysis, the contractor shall pay for the check analysis. All deductions, except moisture, shall be based on the check analysis.

Correction of Gross Weight. (1) Moisture. If the moisture be in excess of the limits specified in the standard analysis, the gross weight of coal shall be corrected by an amount directly in proportion; *e.g.*, if the broken coal as delivered contains 6 per cent. moisture, a deduction of 2 per cent. of the gross weight shall be made. (2) Ash. The weight after correction for moisture shall be reduced at the rate of 1 per cent. for each per cent. of ash in excess of the standard analysis, computed to the nearest tenth of a per cent. (3) Deficiency in thermal units. The weight after correction for moisture shall be reduced at the rate of 1 per cent. for each 100 B.t.u. below the standard heating value, computed to the nearest 50 units. (4) Excess of volatile sulphur. The weight after correction for moisture shall be reduced at the rate of 5 per cent. for each 1 per cent. of volatile sulphur in excess of the standard analysis, computed to the nearest tenth of a per cent. (5) Excess of volatile combustible matter. The weight of coal after correction for moisture shall be reduced at the rate of 2 per cent. for each 1 per cent. of volatile combustible matter in excess of the standard, computed to the nearest tenth of a per cent. (6) Aggregate deductions. After the corrections for moisture all deductions above described shall be totalized, and deducted as a whole.

Excess Clinker shall be cause for condemnation. All coal delivered, in addition to conformance with the standard analyses and other requirements, shall be required to show, after a test of reasonable duration, that it does not produce excessive clinker. When in the opinion of the head of a department, the coal delivered from any mine or group of mines produces excessive clinker delivery from such mine or mines, shall, on notification, be discontinued.

"*Mixed Coal*" shall mean a mixture of different sizes or kinds of coal, the proportions of each size or kind being specified. The ingredients shall be paid for separately. The different kinds or sizes shall comply with the respective standard analyses.

Table 156. Consumption of Gasoline for Pumping*

Gasoline or distillate per effective horsepower per hr., gals.	Approximate consumption of gasoline or distillate in gals per 24 hrs when pumping 100 gals. per min , 100 ft head				
	Total efficiency of pump and drive				
	30 %	40 %	50 %	60 %	70 %
$\frac{1}{10}$	20.0	15.0	12.0	10.0	8.7
$\frac{1}{9}$	22 5	17.0	13.6	11 3	9.6
$\frac{1}{8}$	25.5	19.2	15.2	12.8	10.9
$\frac{1}{7}$	29.0	21.5	17 2	14.4	12.3
$\frac{1}{6}$	33.5	25 0	20.0	16.7	14.3
$\frac{1}{5}$	40 0	30.0	24.0	20.0	17.3

To find approximate number of gals. of gasoline or distillate required per hr. when pumping 600 to 1200 gals. per min., multiply water horsepower by 0.027, or when pumping 100 to 500 gals. per min., multiply by 0.032. For theoretical water horsepower, multiply gals. per min. by total head in feet including friction in pipes, and divide by 4000. 100 g.p.m. = 0.144 m.g.d.

* Byron Jackson Iron Works.

Gasoline. One gallon of gasoline will generate 1 hp. for 8 hrs. or ⅛ gal. gasoline is required for each horsepower-hour. It takes 1⅓ gals. denatured alcohol to do the same work as 1 gal. gasoline.

Gas vs. Steam. Power developed by gas-engine cylinders will lessen at high elevations above sea level, in certain proportion to diam. of cylinder, on account of diminished atmospheric pressure and consequent diminished quantity of oxygen per unit of air. For example: Gas engines of 100 brake-hp. are scaled down to 80 hp., with fixed size of cylinder, under guarantees made for Denver, as against operation of same engine at or near sea level. Small steam pumping plants are extravagant of fuel, while small gas plants are practically as economical as larger, and although application of steam power to pumping is more direct than with gas, great fuel economy of gas-power apparatus enables it to operate at a profit at a much lower mechanical efficiency than the steam engine. For example: A small gas engine geared to a triplex power pump may result in a total mechanical efficiency of 75 per cent.; then, if engine and producer give one brake-horsepower for 1½ lbs. of coal per hr., coal per pump-horsepower will be 2 lbs. Small steam pumping plant with a mechanical efficiency of 94 per cent. requires not less than 4 lbs. of coal per pump-horsepower-hour, and is doing well to accomplish this.

CHIMNEYS*

Chimney Capacity. The temperature of the gases at the base of the chimney should be about 600° F. The frictional resistance of the surface of the chimney is as the square of the velocity of the gases. Ordinarily from

Table 157. Chimney Hight in Feet Necessary for Given Boiler Rating†

Hight in feet	50	60	70	80	90	100	110	125	150	175	200
Diam in inches	Commercial horsepower										
18	23	25	27	29							
21	35	38	41	44							
24	49	54	58	62	66						
27	65	72	78	83	88						
30	84	92	100	107	113	119					
33		115	125	133	141	149	156				
36		141	152	163	173	182	191	204			
39			183	196	208	219	229	245	268		
42			216	231	245	258	271	294	318	340	364
48				311	330	348	365	389	428	459	491
54				363	427	449	472	503	551	594	635
60				505	539	565	593	632	692	748	797
66					658	694	728	776	849	918	981
72					792	835	876	934	1023	1105	1181
78						995	1038	1107	1212	1310	1400
84						1163	1214	1294	1418	1531	1637
90						1344	1415	1496	1639	1770	1893
96						1537	1616	1720	1876	2027	2167
108									2290	2470	2637
120									2827	3049	3255

* Among makers of concrete chimneys, may be mentioned: Weber Chimney Co., Chicago; General Concrete Construction Co., Chicago; Wiederholdt Construction Co., St. Louis; Brick chimneys: H R Heinicke, N. Y. City; M. W. Kellogg Co., N. Y. City; Alphons Custodis Chimney Co., N. Y. City; Bergen and Lindeman, N. Y. City.

† Chicago Bridge & Iron Works, after Kent.

Table 158. Bottom Diameters of Radial-brick Chimneys, Ft.

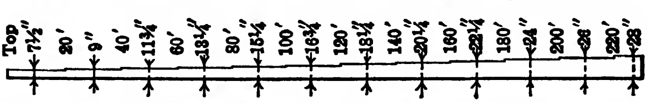
Hight of chimney, ft.	Internal diameter at top						
	3′	3′6″	4′	4′6″	5′	5′6″	6′
	Outside diameters in feet at bottom						
75	7.42	7.69	7.96	8.46	8.96	9.46	9.96
80	7.80	8.04	8.27	8 70	9 13	9.58	10 02
85	8.18	8 38	8.58	8 95	9.31	9.70	10.08
90	8.57	8.73	8.88	9.18	9.48	9.81	10.13
95	8.95	9.07	9.19	9.43	9.66	9.93	10.19
100	9.33	9.42	9.50	9.67	9.83	10.04	10.25
105	9 70	9.78	9.85	10 03	10 21	10 38	10 55
110	10 06	10.13	10.20	10 40	10 60	10 73	10 85
115	10.43	10.49	10 55	10.77	10.98	11.07	11.15
120	10.79	10.85	10.90	11.14	11.37	11.41	11.45
125	11.16	11.21	11.25	11.50	11.75	11.75	11.75
130	.. .	. ...	11.65	11.88	12.10	12.12	12.13
135	.....	...	12.05	12 25	12.45	12.48	12.51
140	....	. .	12.45	12.63	12 80	12 85	12.90
145	. .	...	12.85	13.00	13.15	13.22	13.28
150	. .		13.25	13 38	13 50	13 58	13.66
155		.. .	13 58	13 73	13 87	13 97	14.06
160		..	13 92	14.08	14 23	14 35	14 46
165	. ..	.	14.25	14.43	14.60	14.73	14.86
170	. .	.	14.59	14.78	14.96	15.11	15.26
175	.		14 92	15.13	15 33	15 50	15 66

20 to 30 per cent. of the total heat of combustion is expended in the production of the chimney draft in a marine boiler, to which is to be added the losses of incomplete combustion of the gaseous portion of the fuel, and the dilution of the gases by an excess of air, making a total of fully 60 per cent. The temperature of a furnace is assumed to vary from 1500° to 2000° F., and the volume of air required for the combustion of 1 lb. of bituminous coal, together with the products of combustion, is 154.8 cu. ft., which, when exposed to the above temperature, makes the volume of heated air at the bridge wall from 600 to 700 cu. ft. for each pound of coal consumed.

Reduction of Chimney Draft by Long Flues*

Total length of flues in ft...	50	100	200	400	600	800	1000	2000
Chimney draft, per cent....	100	93	79	66	58	52	48	35

Self-supporting Steel Smoke Stacks may be had of any size; in all cases they should be lined for 30 or 40 ft. above the breeching connection with fire-



Hights of radial brick chimneys and corresponding thicknesses of walls.

* Chicago Bridge and Iron Works, and Wm. Kent. Hp. = 3.33 (Area in sq. ft. − 0.0$\sqrt{\text{area}}$) × $\sqrt{\text{hight}}$, assuming 5 lbs. coal burned per hp. per hr.

Table 158. Bottom Diameters of Radial-brick Chimneys, Ft.—(*Continued*)

Hight of chimney, ft.	Internal diameter at top							
	6′6″	7′	7′6″	8′	8′6″	9′	9′6″	10′
	Outside diameters in feet at bottom							
100	10 75	11.25	11 75	12 25	..			. . .
105	11 03	11.50	11.95	12 40	.		. . .	
110	11 30	11 75	12 15	12 55	.			
115	11.58	12 00	12.35	12 70			..	
120	11.85	12.25	12.55	12 85		..	.	.
125	12.13	12 50	12.75	13.00	13 50	14.00	14.50	15.00
130	12 47	12.80	13.09	13 37	13 80	14.22	14.69	15 15
135	12.81	13 10	13 42	13 73	14 08	14.43	14 87	15 30
140	13 15	13.40	13.75	14.10	14.38	14.65	15.05	15 45
145	13.49	13 70	14.08	14.46	14.66	14.86	15.23	15.60
150	13 83	14 00	14.42	14.83	14 96	15.08	15.42	15.75
155	14 18	14 30	14.68	15 06	15 19	15 31	15 61	15 91
160	14 53	14 60	14 95	15 30	15 43	15 55	15 81	16.07
165	14 88	14 90	15.22	15 53	15.66	15.78	16 00	16 22
170	15 23	15.20	15.49	15.77	15.90	16.02	16.20	16 38
175	15 58	15 50	15 75	16.00	16.13	16 25	16.40	16.54
180		15 80	16 05	16 30	16 40	16 50	16 65	16 80
185		16 10	16 35	16 60	16 68	16.75	16.91	17 06
190		16 40	16 65	16 90	16 95	17 00	17.16	17 31
195		16 70	16 95	17 20	17 23	17.25	17 41	17.57
200		17 00	17 25	17 50	17 50	17 50	17.67	17 83
205			..			17 80	17 98	18 16
210						18 10	18 30	18.50
215						18.40	18 62	18 83
220						18.70	18 94	19 17
225					.	19.00	19.25	19 50

brick, and above that with common brick or cement plaster.—(See E. N., July 20, 1905: Design of Self-supporting Steel Chimneys, by J. D. Adams.)

Radial Brick Chimneys are extensively used, being built of specially formed bricks by companies making a specialty of such construction; they also furnish designs. Chimneys of this type have proved very satisfactory.

*Lining for Brick Chimneys.**—Hight of lining for chimneys is found as follows: For ordinary conditions, where temperature does not exceed 800° F., the lining should be approximately one-fifth the hight of the chimney; above 800° and below 1200°, one-half of the total hight; above 1200° and below 2000°, full hight.

*Protection against Lightning.** For lightning protection two points are the minimum for any chimney of diameter up to 5 ft. Above this, one point should be added for every 2 ft. in diameter or fraction thereof. The points of the conductor should be of copper ¾ in. in diam. by 8 ft. long, with a 1½-in. platinum-covered tip. They should be anchored at the top and extend from the bottom of the corbeling upward. The lower ends of the points are con-

* From specifications of M. W. Kellogg Co., New York, E. N., July 8, 1915.

nected by a copper cable which encircles the chimney. From this loop a $1\frac{1}{2}$-in. 7-strand No. 10 Stubs' gage copper cable is carried down the side of the chimney and connected to a copper ground plate of the three-winged type. On the way up, the cable is anchored every 7 ft. with brass anchors, which support the weight of the cable.

Reinforced Concrete Chimneys* can be built which are entirely safe and practically indestructible. Chimneys now in commission are more or less checked or cracked. This need not condemn them, since reinforcement may be sufficient to hold safely together the blocks formed by the cracks, and yet with the opportunity of introducing steel wherever needed, all cracking ought to be averted, especially as there is always danger that cracks may increase from wind vibration, heat and frost. Concrete and steel, with any degree of heat, expand and contract almost exactly alike. Interior heat affects the shell in another way because concrete is a poor conductor; the inside concrete for an inch or two is heated to a much greater degree than the exterior, and so tends to expand and crack the colder outside. This effect is most marked upon a thick wall. Concrete is an excellent fire-resisting material, although 1500° F. continued for 2 to 3 hrs. will drive out the water of crystallization so as to take away the strength for a depth of $\frac{1}{2}$ to 1 in. Lower temperatures affect it less, and tests at Watertown Arsenal indicate that a good cement mortar will not be appreciably injured at 600° to 700° F. The temperature in an ordinary chimney seldom exceeds 700° F. at the base; 400° to 500° is more usual. E. L. Ransome reports inner shell of a chimney 10 yrs. old of concrete (cement, sand and broken stone), in which he found the hottest part of the chimney opposite the flue perfectly sound and exceptionally hard. When temperatures above 750° are expected, it is safe to employ fire-brick.

A reinforced concrete chimney is so small at the base that it would topple over in a wind if not held by steel. Thickness of outside shell must be sufficient to bear, with steel imbedded, pressures due to weight and to wind. Vertical steel must be inserted all around to resist the pull caused by wind, and steel hoops placed at intervals to stiffen the vertical steel and prevent cracks due to difference in temperature between interior and exterior, and especially to resist vertical shear which corresponds to horizontal shear in a horizontal beam. Assume 50 lbs. per sq. ft. of vertical surface as a maximum wind pressure, corresponding to 100 mi. per hr.; against a curved surface part of it is ineffective, so that the effective pressure against a chimney is not more than 33 lbs. per sq. ft. against a vertical plane whose width is the diam. of the chimney. If the chimney is closely surrounded by buildings, this would of course act only above the roof. Concrete chimneys frequently crack at and above the top of the inner shell, so there is doubt whether this should not be extended higher than the usual one-third. It ought to be possible, however, so to lay and reinforce the concrete as to prevent cracking even with extremes of outside and inside heat. The inner shell must be entirely independent of the outer, as otherwise expansion caused by interior heat will tend to lift and crack the chimney at points of contact. If the inner shell does not extend to the top, the portion of the outer shell where it stops should be especially re-

* S. E. Thompson, Bull. 18, Assn. Am. Portland Cement Manufacturers, 1908.

inforced. Variation in thickness should be avoided if possible, and if necessary should be provided for by additional reinforcement near the outer surface. Changes in section are bad because stresses are introduced which are indeterminate. Horizontal reinforcement should be spaced not more than 12 in. in the lower half of the stack.

Foundations must be designed according to recognized engineering principles. If piles are necessary, they must cover sufficient area so that the line of resultant pressure will fall well within their area at the level where they run into hard ground. Wind causes vibrations which produce repetition of stresses. Following unit values are suggested: Concrete, extreme fiber stress in compression, provided it is capable of attaining strength of 2500 lbs. per sq. in. in 28 days, 600 lbs.; steel in tension, 14,000 lbs.; concrete in shear, 60 lbs. If the sand comes from a bank never used for concrete, it should be submitted to laboratory tests notwithstanding apparent good quality, since it is impossible to pass accurately upon sand by merely looking at it. Instead of mortar of 1 part cement, 3 parts sand, concrete of 1 part cement, 2 parts sand and 2 parts ½-in. stone or gravel will be stronger, denser and more water-tight. Stone up to 1 in. has sometimes been introduced, but although producing a stronger mixture, it is probably less fire-resisting than a finer material.

Concrete has proved suitable for the inner shell as well as the outer, but since mortar is easier to place in a thin wall there is no objection to it. Mild deformed steel gives greater adhesion, but round is safe. With T-shaped steel perfect bond is more difficult because tamping into angles is troublesome.

"Dry" concrete should not be used to imbed steel to take pull. This dry consistence frequently used appears to be the principal cause of failures and large numbers of cracks in many chimneys. Steel must be carefully placed, vertical steel being preferably not far from the center of the wall and horizontal steel outside it. Where special temperature stresses occur, horizontal steel should be as near as practicable to the stressed surface, although not nearer than 1½ in. Steel must be well bonded, and concrete carefully placed and tamped around it. Outside surfaces must be formed by molds, no exterior plastering being permissible. Provide enough horizontal steel to take all vertical shear and to resist the tendency to expansion due to interior heat. Distribute horizontal steel by numerous small rods in preference to larger rods spaced farther apart.

CHAPTER XXIII

DISTRIBUTION RESERVOIRS, STANDPIPES AND TANKS

Size for a Large City (to take care of fluctuations in supply). For 24-hr. periods, the excess consumption is not over 6 per cent. in Manhattan borough, and 9 per cent. in Brooklyn, New York City. The night flow is assumed at 50 per cent. the day flow; 12 per cent. from an equalizing reservoir would prevent undesirable fluctuations. An equalizing reservoir is much more necessary for a grade aqueduct than for a pressure pipe. Large cities usually have much less proportionate variation in consumption throughout the 24 hrs. than small communities. For the latter a much greater percentage of equalizing capacity must be provided, as shown below.

Size for a City of 50,000. Five methods of computing the requisite capacity: (1) In Jour. N. E. W. W. Assn., 1892, p. 49, John R. Freeman recommends a storage capacity of 1,000,000 gals. for each 15,000 population, for 6 hrs., to provide fire protection. (2) In Proc. A. W. W. Assn., 1892, J. T. Fanning reckons the combined fire and domestic consumption for a city of 50,000 as at the rate of 12,000,000 gals. per day. (3) Consider 18 fire streams, each discharging 250 gals. per min., operating for 6 hrs. If the reservoir is to supply this emergency draft, it must give out $250 \times 60 \times 6 \times 18 = 1,600,000$ gals. Assume maximum domestic consumption at the same time. From pumping or other records, plot a curve of hourly variation of consumption; on this pick out the 6-hr. stretch giving the greatest consumption, and compare with the rate per day. It gives say 20 per cent. excess. If the average rate of consumption on a summer's day (or any other season of maximum use) is 8 mgd., the storage required in the reservoir is: 0.25×120 per cent. $\times 8 + 1.6 = 4,000,000$ gals. (in 6 hrs.) (4) Use the method outlined in "Public Water Supplies" (Turneaure & Russell). The rate of fire consumption of a city of 50,000 is 96 per cent. of the average daily consumption, by the rule of Emil Kuichling, Trans. Am. Soc. C. E., 1897. Assuming 8,000,000 gals.* consumption on a summer's day and a daily average, based on yearly records, of 6,600,000, the maximum daily consumption can become $8 \div 6.6 = 120$ per cent. of the average daily consumption. Under (3) the hourly rate for the maximum 6 hrs. in the day might become 120 per cent. of the hourly rate averaged over one day. Therefore, under the most adverse conditions, the consumption for 6 hrs. will be 120 per cent. $\times$ 120 per cent. plus 96 per cent. or 240 per cent. of the average daily supply for a quarter day. $(6.6/4) \times 240$ per cent. $= 3,960,000$ gals. (5) In the report of the National Board of Fire Underwriters on a manufacturing city of 50,000, it is recommended that 8000 gals. per min. be available in the down-town district. For a 6-hr. fire, 2,800,000 gals. would be required. If

* 160 gals. per person per day.

the maximum domestic consumption were occurring at the same time, the reservoir supply must be 2.8 + 2.4 = 5.2 million gals. in 6 hours.

Size of Standpipe Required. For cities and villages, be sure to get the tank large enough. There should be at all times enough water stored to furnish several fire streams and supply the consumption when the pumps are not running. With a liberal allowance for increase, 30 gals. per inhabitant should be the minimum; no tank should have a capacity less than 30,000 gals. For institutions, make a careful investigation of the water used and allow a large excess for fire protection. The capacity and hight for fire protection to factory buildings is generally prescribed by the insurance companies. The largest elevated tank ever built (1912) is at Louisville, Ky. It has a capacity of 1,200,000 gals.; hight of 220 ft. from the top of foundations to top of tank. The tank is 50 ft. diam., and 90 ft. deep, with a 48-in. riveted-steel riser pipe. This suggests the possibility of similar tanks on high towers for fire protection in congested districts of cities. Chicago Bridge and Iron Works state that they are prepared to construct tanks of much greater dimensions than the Louisville tank. Where a tank can be located on a natural elevation, it is economical to make the diam. 10 to 20 per cent. greater than the hight; as a rule the hight should not exceed 60 to 70 ft.

DISTRIBUTION RESERVOIRS

Reservoir Floors. In the Queen Lane reservoir, Philadelphia, sheet lead expansion joints were put in the walls and floors every 40 ft. This flexible metal could be bent inside the forms while concrete was being placed on one side of the joint. The joint proved satisfactory, there being no leakage. The lead sheets were 6 in. wide over all, $\frac{3}{32}$ in. thick, of length to suit; $\frac{3}{4}$-in. holes 6 in. apart promoted adhesion.—(E. R., Apr., 1912, p. 460.)

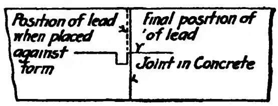


Fig. 295.—Queen Lane expansion joint.

The concrete floor of Mt. Hope reservoir, Rochester, N. Y., is in two layers, the lower 3 in. thick, and the upper one 8 in., with waterproofing of 6 layers of coal-tar and 5 layers of tar felt alternating between. Just below joint in upper layer the lower concrete layer is made 6 in. thick for 1 ft. width, and reinforced across the joint by $\frac{1}{4}$-in. rods bent U-shaped. Elsewhere than under the upper joints the lower floor was allowed to crack. That cracks would form in the lower floor under the upper joints unless reinforcement was used, was proved by an experiment, intended to imitate actual conditions, including the stresses from 30 summers followed by 30 winters. This showed definite cracks in the lower floor; but the waterproofing, with the exception of one weak spot, remained intact.

Reservoir at Vancouver, B. C., was founded on rock. The floor and side slopes were covered with $4\frac{1}{2}$ in. of concrete. The floor slabs do not exceed 12 ft. square, with V-shaped joints, filled with clay or asphaltum. The side slopes, 1 on 2, were lined with reinforced concrete, with expansion joints every 50 ft. The reinforcement consisted of Clinton wire cloth of two weights, the lighter for the upper portion of the slopes. The slope concrete was

anchored by 2-in. × ¼-in. anchor bars, 2 ft. 6 in. long, 5 ft. centers, driven into the bank. The expansion joint is shown in Fig. 297.—(H. M. Burwell, Eng. Contr., Jan. 4, 1911.)

A good water-tight reservoir lining can be made of properly proportioned asphaltic concrete, a few inches thick, similar to that used for highway surfacing.

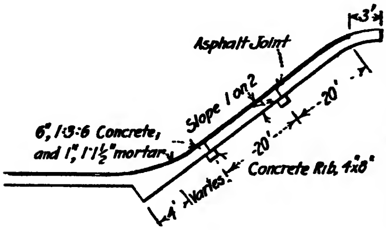


FIG. 296.—Joints in lining of Beacon Hill reservoir, Seattle. (Eng. Contr., Apr. 7, 1912)

Slope paving of Central reservoir, Oakland, Cal., is 6 in. thick (1:2.5:5 concrete) laid in 12-ft. squares, with expansion joints made of "D" grade asphaltum mixed with carbolated lime or marble dust, and run hot, on slope of 1 on 2.—(E. N., Jan. 5, 1911, p. 15; E. R., Oct. 1, 1910, p. 375.)

Many distribution reservoirs have failed because the weight of the water cracked the bottom lining. Great care should be taken. At Portland, Oregon, the concrete lining was 5 to 6 in. thick and reinforced with ⅜-in. sq. twisted rods, 2 ft. apart; it cracked before filling, and more cracks developed after filling.—(W. R. Hill, Proc. A. W. W. A., 1902, p. 23.)

Concrete slope lining, Hill View Reservoir, New York, was placed in strips 8 ft. wide extending from toe wall to berm 21.5 ft. above, slope 1 on 2,

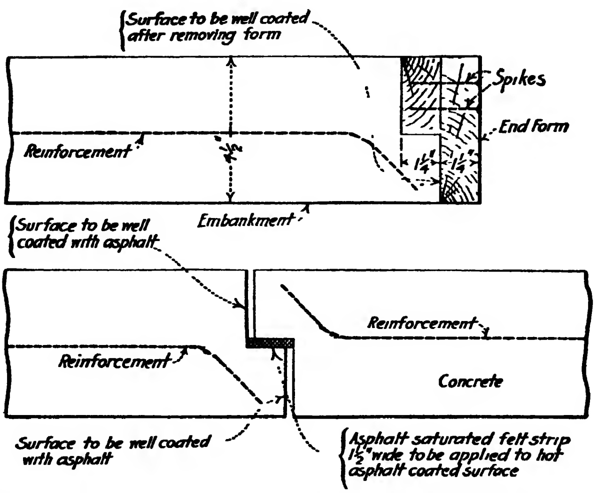


FIG. 297.—An expansion joint for concrete reservoir linings, Vancouver.

length on slope 48 ft., thickness 8 in. Alternate strips were first placed between timber side forms about 4 × 8 in. secured to stakes and later the intermediate strips were filled in. Concrete, moderately wet, was deposited by 1 yd. buckets directly on the slope, at the upper limit of the portion of the space to be filled by each batch; it was roughly brought to thickness by

means of a wooden screed traveling on the side forms or the edges of previously cast strips. Surface finishing was done with long-handle, square-point shovels and great smoothness was not attempted nor regarded as necessary. To keep water from the berm and slope above out of the joints each strip was dished slightly so that the water would flow down its center. The edges of each strip were thus slightly raised at the joints, and along each joint six or seven tapered round wooden plugs 2 to 3 in. in diam. were set to form weep holes through the strip to permit the escape of any water getting under the strip at the berm joint. (See also p. 204, Fig. 101.)

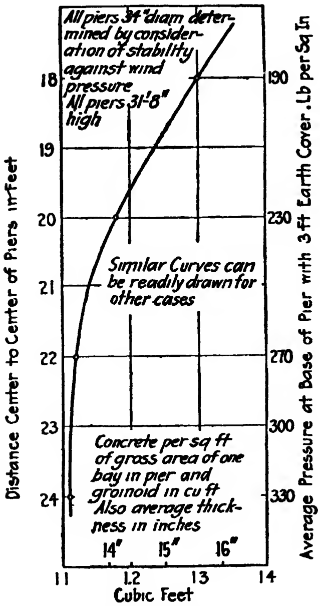


FIG. 298.—Economic span for groined arches.

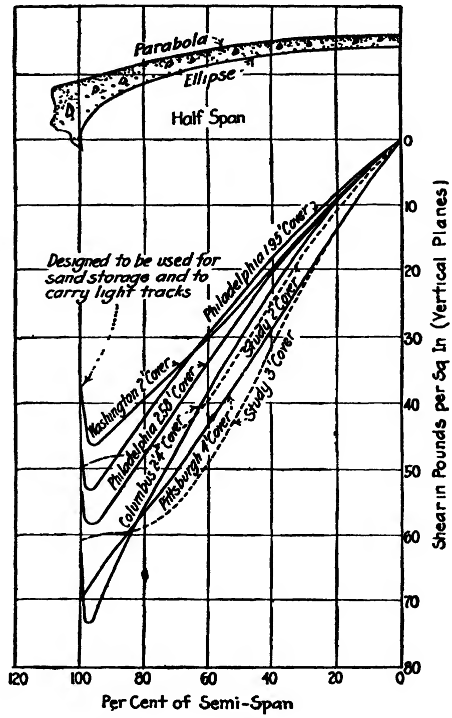


FIG. 299.—Shear in existing groined arches (100 lb. per square foot live load).
(See page 522)

Repairing Lining. Leakage from a small concrete-lined reservoir near Biddeford, Maine, was effectually reduced by cleaning the surface of the concrete so as to expose the cracks and then cutting out the cracks, for a depth of a few inches, to sufficient width to permit calking. Joints so prepared were thoroughly calked with oakum by ship-calkers, flushed with hot tar and surplus tar burned off with plumber's torches. The whole surface of the concrete lining was then given 3 good coats of heavy Portland cement grout, brushed on, having a total thickness of about ½ in. Leakage was reduced from 2½ mgd. to 0.1 mgd. In pointing masonry to be exposed to water, hardwood calking tools may be used to drive mortar thoroughly into joints.

Covered Reservoirs. *Effect on Water.* At Ridgewood (Brooklyn, N. Y., waterworks), ground water, or mixed ground and surface waters, cannot be stored in open reservoirs on account of growth of offensive microörganisms. Artificial storage reservoirs usually are unnecessary for a properly designed ground water-system, because pore spaces of sands and gravels form natural reservoirs of much larger capacity than could be obtained in artificial basins. Such storage does not invite evaporation or growth of organisms. (See p. 683.) Many subsurface and filtered waters cannot be kept in open reservoirs without deterioration, and so covered reservoirs are necessary for them.

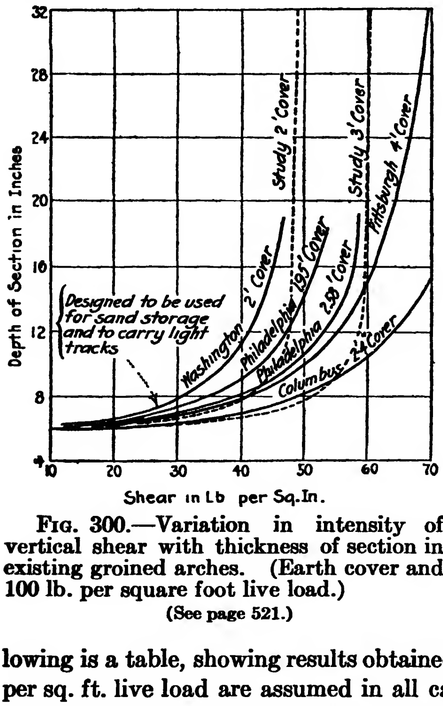


FIG. 300.—Variation in intensity of vertical shear with thickness of section in existing groined arches. (Earth cover and 100 lb. per square foot live load.)
(See page 521.)

Groined Arches. *Method of Comparison.* Assuming that this type of structure stands up through arch action, and not cantilever, and that joints are formed along the diagonals, each pier would be the abutment for 4 arches, the halves of which would be trapezoids in plan. By drawing a line of resistance and computing stresses in concrete, this method may be used as a comparison, although it is understood that the stresses are not necessarily those which obtain. Following is a table, showing results obtained for various groined arches; 100 lbs. per sq. ft. live load are assumed in all cases. For detailed methods of computing, see Jl. N. E. W. W. Assn., Vol. 19, 1905. Also "The Groined Arch in Filter and Covered Reservoir Construction," T. H. Wiggin (National Ass'n. of Cement Users), E. N., Apr. 7, 1910. For sections, see Fig. 383, page 738.

Table 159. Comparison of Groined Concrete Arches *

No.	Date	Location	Engineer	Depth of cover	Concrete mixture	Clear span	Rise	Crown thickness
				ft.		ft.	ft.	in.
1	1903	Philadelphia, Torresdale.	J. W. Hill.	1 95 2.58	1:3 :5	14 00	3.00	6
2	1905	Columbus.	J. H. Gregory.	2 33	1:2.5:5.5	15.16	3.16	6
3	1903	Washington, D. C.	Col. A. M. Miller Allen Hazen.	2 00	1 :2.9:5	15.54	3.50	6
4	1904	Pittsburgh.	Morris Knowles	3.00–4.00	1 :3 :5	15.75	4 00	6
5	Proposed.	Jerome Park, New York.	Wm. B. Fuller.	2 00	— — —	18.00	4.50	6
6	Proposed.	Study, B. W. S, New York.	J. W. Smith.	1 5–3.00	1 :3 :5	19.16	5.00	6

* See also E. R., Nov. 15, 1913, p. 359.

Table 159. Comparison of Groined Concrete Arches.—(*Continued*)

No.	Depression over pier	Pier section	Volume saved by depression		Mean thickness of roof	Max. pressure at pier base, lbs. per sq. in. incl. 100 lb., per sq. ft live load	Hight of piers	Arch stresses, lbs. per sq. in. (compression)	
			Cu. yds.	Average depth				Crown	Joint of rupture
	in.	in.		in.	in.		ft. in.		
1	21	22 sq.	2.70	3.5	7 2	265	14–4	122	259
2	23	20 sq.	3.36	3 85	7.0	330	11–8	134	262
3	24⅝	30 sq.	4 0	4 0	7.9	180*	25–7	115	234
4	26	27 d	4 33	4 33	8 25	380	21–6	163	267
5	30	24 d	6 2	5 0	7 75	410	23–0	———	———
6	31	34 d.	9.25	6 2	8 3	330	31–8	165†	294
								131‡	233

* Roof designed to be used for sand storage † 3-ft load ‡ 2-ft. load

REINFORCED CONCRETE STANDPIPES

Design and Construction.* Common practice has been to figure bursting pressure by "thin cylinder" formula and provide ring steel sufficient at from 12,000 to 16,000 lbs. per sq. in. for all tension, thus disregarding tensile strength of concrete and practically necessitating fine cracks due to stretch of steel. Rings have been of plain or deformed rods, tees, or other shape; ends lapped, fastened with clips or clamps or similar devices, or welded; one or two rings in width of wall, according to needs and judgment of designer. Vertical steel of a variety of shapes has been used to support rings and resist moments, shears and other stresses. Additional vertical and bent rods have been placed in junction of walls with base, or floor, extending a few feet up into walls, to provide for special stresses here due to differences in function and strain of walls and floor; quantity and arrangement of this steel has been almost wholly a matter of judgment. Spacing of rings and size of metal in them have been varied in the vertical according to decrease in stress, from bottom to top, but some minimum size of metal and maximum spacing have been used for some set distance below top. Thickness of walls has been made sufficient to envelop all steel thoroughly and, according to judgment of designer, to be water-tight under pressures corresponding to depths, with minimum of 6 to 12 in. at and near top, depending largely on the diam. So-called waterproofing materials have frequently been incorporated in the concrete, and coatings applied to interior surfaces of the standpipe. None of these should be needed; experience has shown them of doubtful benefit; hydrated lime has increased efflorescence and consequent disfigurement. Effectual means are: skillful selection and gradation of aggregates to secure maximum density; liberal use of cement; abundance of mortar; sufficient water to make concrete wet enough to flow into and completely fill small spaces where stiff concrete cannot be (or is not likely to be) forced by tamping; very thorough mixing; tight forms; skillful joggling of all concrete as deposited; keeping thoroughly wet from time forms are removed until concrete is 2 weeks or more old; protecting new concrete from hot sun; using cement which develops minimum tem-

* Based partly on "A New Theory of Concrete Reservoirs" by H. Andrews, Jl. A. E. Soc. June, 1911.

perature during setting and hardening; thorough removal of laitance, dead cement and dirt, especially at joints; scrupulous inspection. Inevitable interruptions for erecting forms, placing steel, on account of bad weather, etc., should be as brief as possible. (See also p. 610.)

Recently experience and investigation have indicated that it is advisable not to stress beyond the tensile strength of concrete structures to contain water under pressure, especially important permanent structures. This consideration has led to very rich concretes, such as 1:1½:3, 1:1:2; working stresses of 3000 to 6000 lbs. in the steel; thicker concrete walls, and tensile stresses allowed on concrete of 300 to 500 lbs. As there is a tendency for the diam. of a reservoir to increase after it is filled with water, due to the elasticity of the steel in tension, and as the base of the reservoir is practically rigid, due to contact with foundation, some reinforcing material extending from the base up into the walls to take care of the bending moment and shear at the base has been necessary. There is no exact method of obtaining the amount of steel required here, but it has been underestimated. In one small standpipe no connection was made between wall and base, but instead a smooth sliding joint was provided which has proved watertight and has been satisfactory in other respects.

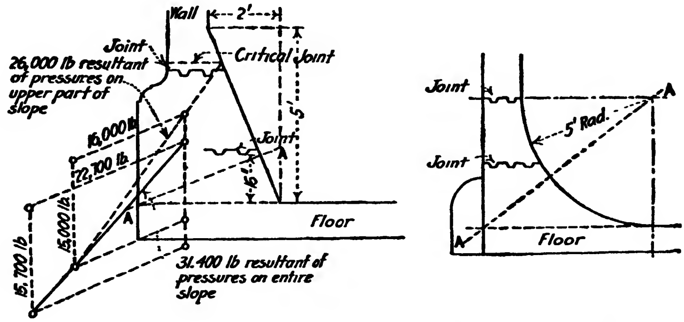
As building forms and placing steel preclude making the concrete work continuous, make as short intervals as possible between successive layers, and bond in the best possible manner. Instructions outlined were followed in constructing reservoirs at Waltham, Manchester, and Lisbon Falls (all in New England States) except that the latter two were made with 1:1½:3 concrete plus 5 per cent. hydrated lime. For Manchester reservoir the thickness of the wall at base is 20 in., and at top 12 in. The steel is designed for a unit working stress of 12,000 lbs. per sq. in. when the reservoir is full. At the base are rods embedded in the floor, 1 in. diam., 12 in. on centers, extending into the wall about 4 ft. 6 in. The floor was finished with 1-in. granolithic, and the walls plastered two coats, making about 1 in. of 1:1 cement mortar. There developed at several horizontal joints—especially three lower joints—between days' work, some seepage of water, which in three places increased to positive leaks. Seepage at upper joints gradually stopped, presumably due to filling of pores by hydrated lime. Examination of reservoir after emptying showed horizontal cracks at joints mentioned, about 30 ft. long, extending through the wall, also vertical cracks in the plastering extending upward 20 ft.; furthermore, there were checks in the plastering from which water oozed back into the reservoir after it was emptied. From these observations, were drawn the following deductions: (1) not enough vertical steel properly disposed to distribute fully the bending moment and shearing stress between the rigid base and the walls; (2) the ultimate strength of the concrete was probably exceeded when the reservoir was filled, thus producing vertical cracks; these vertical cracks allowed the water to permeate the walls to the lines of least resistance, the horizontal joints; (3) a rich plaster coat on more or less permeable concrete was useless, as the usual crazing and the vertical cracking would allow percolation of water through it. From tests on concrete beams it has been found that microscopic cracks developed on the tension side when the steel reinforce-

ment was stressed to 4000 or 5000 lbs. per sq. in., or perhaps less; these cracks gradually widened until visible. Only remedy seemed to be to make walls so thick and of such composition that tensile strength would never be exceeded. In the reservoir for Rockland, Mass., especially rich concrete was used; thickness of wall which would insure that ultimate tensile strength of concrete would not be reached when reservoir was filled; increased vertical reinforcement especially between base and walls; and a steel dam at each horizontal joint between days' work to prevent direct seepage through joint. Hydrated lime was omitted, as proposed density of concrete did not require it for impermeability; where it had been used it had caused unsightly efflorescence wherever there had been seepage. Plastering was omitted, but instead, three coats of soap and alum solution were applied.

Manchester, N. H. (R. C. Allen, Engr.), standpipe leaked only on about 20 ft. on the easterly and southeasterly sides, where maximum temperature stresses occur from the sun, while in every other place there was not a drop. The leakage between the first and third joints was estimated 15,000 gals. maximum in 24 hr. That was the most serious leak; the leaks at other times have been matters of perhaps a few hundred gallons a day. In 1911 not more than 10 or 15 gals. per day was seeping out in the space 8 ft. long and 6 in. wide at the very base. L. C. Wason: The Attleboro, Mass., standpipe (50 ft. diam. 100 ft. high to water-level) was designed on the basis that the hoops take the entire load with a unit stress in steel of 13,500 lbs. per sq. in. Not to have the bars too close together, $1\frac{1}{2}$-in. bars were used in 2 rows from the bottom to 61 ft. From 61 to 81 ft., a single row of same size bars was used; from 81 to 100 ft., $1\frac{1}{8}$-in. bars. In the upper 15 ft. steel was kept a constant, although hoop stresses from the water were constantly diminishing, to provide for possible stresses by ice. The wall thickness of 18 in. at the bottom tapered to 8 in. at the top. To space the bars, 4-in. channels with $\frac{3}{8}$-in. holes through both flanges, were set upright at intervals of 15 ft. with the web radial, a $\frac{1}{4}$-in. rod was passed through the holes, and the hoops rested directly on the end of these $\frac{1}{4}$-in. rods, which were then bent up to secure the hoop firmly. From 61 ft. to the top, 3-in. channels were used. The floor was 12 in. thick, and met the wall with a curve of radius, 5 ft. The top of the floor was reinforced with $\frac{1}{4}$-in. square-twisted bars, 6 in. centers each way, carried well up the curved corner, and into the wall; $\frac{5}{8}$-in. square-twisted bars were also placed radially at intervals of about 3 ft. around the circumference, the ends projecting up into the wall 10 ft. The foundation slab, 18 in. thick, was increased immediately under the walls to 4 ft. for a width of 5 ft. A concrete curb 3 ft. high, 12 in. thick, with a curved top, was built around the outside of the standpipe bottom, but not monolithic with it. Bars were obtained long enough so that 3 would reach entirely around, with a lap of 40 diam. at each joint. Two wire rope clips were used at each splice to insure bars being held firmly together. By the test at the Watertown Arsenal, these clips alone were sufficient to insure full working stress of the bare bars; any additional bonding stress from the concrete was an added factor of safety. Splices were staggered so that the eleventh laps came in the same vertical plane. Unless very low stresses are used in the hoops, or vertical steel bars are added,

there may be a distinct movement on some joint, as is indicated by the fact that when a standpipe has been kept full of water for some time, leakage through these joints almost entirely disappears, but on emptying and refilling, the leakage occurs as vigorously as at first.

In the Westerly standpipe, the steel stress in the lowest foot was 6000 lbs. per sq. in., increasing 1000 lbs. for each foot in hight until a maximum stress of 12,500 lbs. was reached. No vertical steel was used; on filling, a small line of dampness occurred in a horizontal joint about 3.5 ft. above the floor. Fig. 301 shows sections of the bases of the two standpipes, and the position of the so-called critical joint. On the section of the Westerly standpipe, the diagram indicates the forces from the weight of the wall and the outward pressure of water on the sloping corner. Projecting the resultant backward to the inside of the wall, a point is found where compression changes to tension; this is the critical joint. It is very close to the joint where the second day's work left off. This is the joint that leaked. After the tank had been filled three weeks the leakage was just sufficient to dampen the surface. In addi-



Westerly, R. I. Attleboro, Mass.

FIG. 301.

tion to the leak near the bottom three spots appeared: one very close to the floor, caused by a breakdown in the mixer, resulting in a delay of an hour or so; another 23 ft., and one 40 ft. up. These were grouted under high pressure and stopped. The worst of the three, before grouting, leaked one drop in 6 sec. Very great caution was exercised in finishing the joint between days' work. Two men were kept on into the night after concrete was placed to scrub the top surface carefully with brushes and water to get the laitance off and wash the surface of all stone showing on top as clean as before being mixed, because cement bonds well to clean aggregate, but does not bond well to old cement. Roughing strips were put in the wall to form grooves. These were also removed in the evening. Before concrete was placed the following morning the surface was thoroughly washed with water, then coated with neat cement, rubbed in with a brush, and grooves were filled with neat cement.

At Bridgewater the floor ran straight across to the outside with just a little offset downward and then a roughing strip. Then the wall was cast, dovetailed into the floor and a little fillet of concrete was placed between the floor

and wall and worked in with a great deal of care. Water was put in 10 weeks after the foundations were started; the last concrete placed was then about 1 month old. Prison labor, not skilled in concrete, was used; after 1½ years' service, the tank is without a leak (1911) and has been absolutely dry from beginning to end. Three times as much steel was used at the base as at Manchester (suggested as reason for tightness); one set of rods carried 5 ft. high; one set, 10 ft., one set, 15, and another set to the top.

Wason found it neccssary to place the steel within 2 in. of a surface to be protected from cracking; less than 1 in. is very much more effective; if the steel is more than 2 in. from the surface, it is wasted. Messrs. Weston and Sampson believe that Andrews' theory covers few cases and is of limited application. It will not do for large standpipes: the cost is too high.

At Westerly, R. I., concrete was mixed to make only 12 cu. ft. to a 4-bag batch, of 20 parts by weight of cement, 40 parts of No. 4½ sand and 40 parts of No. 3 stone, screened to remove everything below ½ in. This mixture had the maximum density of all measured and appeared satisfactory. The curve given by this mixture, however, on volumetric tests, was nearly 10 per cent. higher than the "Ideal"* at the point of tangency. With the densest mix, the addition of hydrated lime increases the volume almost directly in proportion to the lime used, and therefore is of little use. In the Waltham standpipe a small leak was stopped with lead wool by an ordinary workman. The deformation of steel stressed in excess of 4500 lbs. per sq. in. exceeds the ultimate elongation of plain concrete; the construction of tanks in which the steel is stressed much higher, involves cracking. The Attleboro tank was kept filled with water during construction to a level of about 20 ft. below where concrete was being placed—principally as a protection to the workmen. When the tank was about 6 yrs. old, a successful attempt was made to stop several leaks by grouting. On the assumption that repairs should be made when the wall was stretched to the maximum limit, the work was done with the tank full. At each leak a hole, larger on the inside, was cut exposing the steel, filled with small crushed stone, held in place by a piece of wire netting, and the whole plastered over with mortar, a short length of ¾-in. pipe passing through this plaster. The steel when exposed was bright and clean. Neat grout was poured into an 18-in. length of 6-in. wrought-iron pipe with flanged top and bottom. From the bottom a lead pipe connected with the pipe in the wall, and to the top a large carbonic acid tank under 3000-lbs. pressure was connected. By a valve any pressure could be brought on the grout, shooting it into the hole back of the hardened plaster facing. As much as ½ cu. ft. was forced in at one point. In the Attleboro tank there have been, from time to time, a few small leaks, which indicates that 1:2:4 concrete, properly placed, can be made watertight, although it may be better judgment to use a richer mixture. Because of the contraction of concrete in setting, it is in tension and the steel in compression. A first principle of this class of construction is to proportion the parts so that tensile strength of concrete shall not be exceeded.

* Taylor and Thompson, "Concrete, Plain and Reinforced," 1909, p. 776.

Experiences. Reinforced concrete standpipes over 50 ft. high in localities having much freezing weather were not regarded with favor in a general discussion by New England W. W. Assoc., March, 1915. At Westerly, R. I., leaks soon developed at construction joints and more leaks developed every time the standpipe was emptied and filled. Hot paraffin waterproofing was unsuccessful, as was plastic slate, which latter gave a bad taste to the water. Finally, 1911, whole inside was covered with 5 layers of felt saturated with a bituminous compound alternating with coats of a bituminous paint, costing $1782. Effect of ice not known as water has uniform temperature of 54° F. Attleboro standpipe was given 13 coats of Sylvester wash,* with unsatisfactory results, and subsequently was waterproofed like the Westerly standpipe, at a cost of $3000. Some spalling on outside, and patching was so unsightly that structure has been enclosed in an 8-in. brick wall.† At Waltham an unsuccessful attempt was made to waterproof by applying a brush coat of tar. At Manchester, Mass., a large crack occurred as soon as standpipe was filled; seepage at many points. Attempts were made to repair cracks by covering with lead plates, but eventually an asphalt waterproofing was the only satisfactory solution. Outside spalling here and at Waltham. At Lexington an unsuccessful attempt at waterproofing by plastering on a coating of canvas resulted in the accumulation of the canvas in the bottom of the standpipe. H. B. Andrews, of Simpson Bros. Corp., Boston, had had no difficulty from seepage,

Table 160. Costs of Reinforced Concrete Tanks and Standpipes

L. H. Allen, Aberthaw Construction Co. (Cement Age, Sept., 1911)

Location	Diam., ft.	Hight, ft.	Cost	Cost per 1000 gals.	Remarks
Waltham, Mass....	100	43	$25,785	$12 90	Concrete roof on steel trusses; center column.
Leicester, Mass......	80	30	12,380	11.00	No roof.
Paris, Me............	80	14	7,150	13 60	No roof.
Manchester, Mass....	60	72	36,000	23.60	
Attleboro, Mass......	50	100	36,000	24.50	
‡Douglass Mass......	46	20	5,260	21.15	
Westerly, R. I........	40	70	16,000	24.30	
Warren, R. I........	40	50	10,590	22.50	
‡Walker and Pratt, Watertown, Mass.	35	20	3,275	22.75	
‡Gilbert and Bennett.	30	70	9,000	24.30	
‡Gilbert and Bennett.	30	30	8,000	50 00	On tower 40 ft. high
Burlington, Vt.......	30	30	5,050	31.40	On tower 25 ft. high
Littleton, N. H.......	27	27	2,080	18.10	
Madison, N. J........	25	130	16,000	33.40	Around old steel pipe, 75 ft. high.
‡Vineyard Haven., Mass.	20	70	6,000	36.60	
Danbury, Conn......	20	30	7,250	103.00	On tower 25 ft. high
Medfield, Mass......	20	23	1,530	27.80	
Medfield, Mass......	20	20	1,400	29.20	
Littleton, N. H......	17	17	1,080	38.60	
Fall River, Mass....	16	15	850	38.60	

*In first 35 ft., 9 coats at top. See page 611 for description of Sylvester's wash
† With air space between and weep holes in bottom of brick wall.
‡ Engineers estimate; tanks not built. See also Jl. Assn. Eng. Soc., Jan., 1911, and Jl. N. E. W. W. Assn., June, 1915.

however, in some standpipes erected; in most recent structures he had increased richness of concrete, mixture now being 1:1:2; none was over 60 ft. high. Sunny sides of standpipes had developed most cracks and leaks. Opinion was expressed that in warmer climates results were and should be satisfactory.—(E. N., Mar. 18, 1915.)

Prevention of Joint Leakage. To prevent leakage through horizontal joints between pourings of different days, grooves can be formed at close of a pour to receive the section above. A better method is to use copper strips 4 to 6 in. wide, bent at right angles on both edges ½ in., and with a small V crimp in the center, pressed into the fresh concrete one-half their width, with end joints lapped 3 to 6 in. so as to form continuous strips around the tank. Flat steel strips about ¼×6 in. have also been used successfully. If properly embedded in concrete, well made, metal will probably last indefinitely. To thicken the side walls at junction with the floor and tie the floor to the walls with steel is a mistake. The floor should be treated as unit in itself. By filling all joints with asphalt, they will be waterproof under any head, or even settlement of the wall. To prevent cracks, due to elongation of rods on filling of tank, reinforcement should be increased at bottom.—(E. N., Sept. 7, 1911.)

Since 1911, Simpson Bros., Boston, have built following tanks:

Location	Capacity, gals.	Diam., ft.	Hight, ft.	Date
W. Falmouth, Mass..	402,000	40	40	1913
Lexington, Mass......	550,000	30	104	1912
Chelmsford, Mass...	203,000	45	16	1913
Brunswick, Me.	2,500,000	97	46	1913
W. Brookfield, Mass	251,000	50	20	1913
Woonsocket, R. I ..	1,646,000	79	42	1913
Jamestown, R. I	360,000	35	50	1914

Waterproofing New Ulm Concrete Reservoir. A 1,000,000-gal. reservoir, New Ulm, Minn., is 75 ft. diam., 30 ft. deep, reinforced concrete, with reinforced concrete conical roof. Hard clay, well drained naturally, with 12-in. layer of stone, voids filled with wet, fine-grained concrete, formed the foundation for a 10-in. floor slab, reinforced near its surface with 16-gage 3-in. mesh expanded metal. Concrete aggregate was carefully graded, pebbles from ¼ in. to 2½ in.; 20 lbs. of hydrated lime to 1 bbl. of cement were added to reduce permeability. Walls were brushed and two coats of 1 : 2 cement plaster was applied, containing about 10 per cent. waterproofing compound. Slush coat of 1 : 2 cement mortar was applied to floor and a brush coat of cement grout applied to this. When water was admitted, leakage took place through half the length of a circular seam, about 4 ft. from the wall, between old and new concrete; the foundation and 1 ft. of wall had been poured 7 days before laying the center of the floor; force exerted by the load on the foundation of the walls, opened the bond. To remedy this, about 2½ in. was cut out of the seam and filled with 1 : 1 cement mortar. This failed because difference in floor pressure between empty and full reservoir caused a movement which would not occur in side walls. The added stress opened the seam still further.

Reservoir was next drained and stood empty 90 days, to drain the clay foundations thoroughly. All visible weak places were cut out to about 2½ in. deep and entire surface given two brush coats of "Ironite" and cement mixed in proportion 1 to 5. Parts cut out were then filled with strong cement mortar, walls were plastered with 1:2 cement mortar, 10 ft. high, waterproofed above, and an additional layer of reinforced (¾-in. and 1-in. round steel rods, in circles and radially) concrete, 6 in. at walls and 4 in. at center, added. Whole surface was then brush coated with waterproofing compound. After standing 10 days the reservoir was filled, at the rate of 2 ft. a day, until depth of 20 ft. was reached; then 1 ft. a day till full. Close watch and measurements showed no leakage.—(E. R., Dec. 17, 1910.)

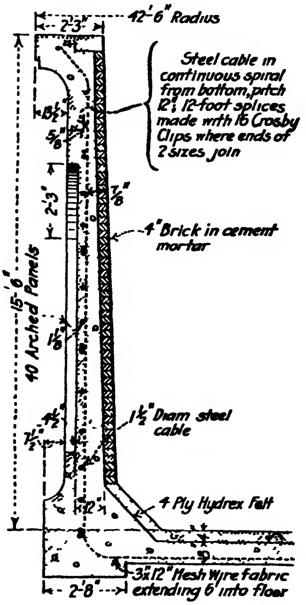


FIG. 302.—Cos Cob reinforced-concrete reservoir. Wall section.

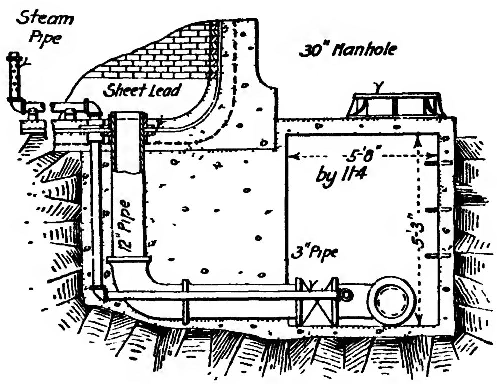


FIG. 303.—Cos Cob reservoir. Section of valve chamber. (Connecticut)

(Founded on rock Capacity, 600,000 gals Built by Westinghouse, Church, Kerr & Co. for N. Y., N H. & H R R. E R., Sept. 21, 1907.)

STEEL TANKS AND STANDPIPES

Standpipes were much used until about 1910, but owing to their greater cost, and the great danger of failure in high standpipes, there are but few built now. Both wood and steel have been used for elevated tanks, and for the substructure. Stone and brick are occasionally used, but usually at a greater cost.

Specifications for Elevated Steel Tanks on Towers.* *Loads.* Dead load shall consist of weight of structural and ornamental steelwork, platforms, roof, piping, etc. Live load shall be contents of tank, movable load on platforms and roof, and wind pressure. Load on platforms and roof assumed at 30 lbs. per sq. ft., or a 200-lb. concentrated load applied at any point. Wind pressure 30 lbs. per sq. ft., acting in any direction, surfaces of cylindrical tanks being calculated at two-

* G. W. Birch-Nord, Trans Am. Soc. C. E., Vol. 64, 1909.

thirds of diam. multiplied by hight. Similar assumptions may also be made for spherical and conical surfaces by using correct hights. Live load on platforms and roof shall not be considered as acting together with wind pressure.

Unit Stresses: All parts of structure shall be proportioned so that sum of dead and live loads shall not cause stresses to exceed:

Tension in tank plates	12,000 lbs. per sq. in. of net area
Tension in other part of structure	16,000 lbs. per sq. in of net area.
Compression	16,000 lbs. per sq in. reduced
Shear on shop rivets and pins	12,000 lbs. per sq. in.
Shear on field rivets (tank rivets) and bolts	9,000 lbs. per sq. in.
Shear in plates	10,000 lbs. per sq in. of gross area.
Bearing pressure on shop rivets and pins	24,000 lbs. per sq. in.
Bearing pressure on field rivets (tank rivets)	18,000 lbs. per sq in.
Fiber stress in pins	24,000 lbs. per sq in.

For compression, unit stress shall be reduced by formula:

$$p = 16{,}000 - 70\frac{l}{r}.$$

p = permissible working stress in pounds per sq. in.; l = length of member, from center to center of connections, in.; r = least radius of gyration of section, in.; $\frac{l}{r}$ shall never exceed 120 for main members and 180 for struts and roof members. Stresses due to wind may be neglected if less than 25 per cent. combined dead and live loads. Unit stresses in bracing and other members taking wind stresses may be increased to 20,000 lbs., except as shown in preceding sentence. Pressures permissible on bearing plates:

Brickwork with cement mortar	200 lbs. per sq. in.
Portland cement concrete	350 lbs. per sq. in.
First-class sandstone	400 lbs. per sq. in.
First-class limestone	500 lbs. per sq. in.
First-class granite	600 lbs. per sq. in.

Details of Construction. Plates forming sides of cylindrical tanks shall be in courses of alternately large and small diam., so that courses shall lap over each other, inside and outside, alternately. Joints for horizontal seams, and for radial seams in spherical bottoms, shall preferably be lap joints. For vertical seams double-riveted lap joints shall be used for $\frac{1}{4}$-, $\frac{5}{16}$-, and $\frac{3}{8}$-in. plates. Triple lap joints shall be used for $\frac{7}{16}$- and $\frac{1}{2}$-in. plates; double-riveted butt joints shall be used for $\frac{9}{16}$-, $\frac{5}{8}$-, $\frac{11}{16}$- and $\frac{3}{4}$-in. plates, and triple-riveted butt joints for $\frac{13}{16}$-, $\frac{7}{8}$-, $\frac{15}{16}$-, and 1-in. plates. $\frac{5}{8}$-in. rivets shall be used for $\frac{1}{4}$-in. plates; $\frac{3}{4}$-in. for $\frac{5}{16}$-in. plates; $\frac{7}{8}$-in. for $\frac{3}{8}$- to $\frac{7}{8}$-in. plates, inclusive, and 1-in. rivets for $\frac{15}{16}$-in. and 1-in. plates.

Rivets shall be spaced so as to make the most economical seams (70 to 75 per cent. efficiency). In no case shall spacing between rivets along calked edges of plates be more than ten times thickness of plates. All rivets shall be entered from inside of tank, and shall be driven from outside, that is, new heads on rivets shall always be formed from opposite side of plate to which calking is done. Plates $\frac{3}{8}$ in. thick, and not more than $\frac{7}{8}$ in. thick, shall be subpunched with punch $\frac{3}{16}$ in. smaller in diam. than nominal size of rivets, and shall be reamed to finished diam. not more than $\frac{1}{16}$ in. larger than rivet. Plates thicker than $\frac{7}{8}$ in. shall be drilled. Minimum thickness of plates for cylindrical part shall be $\frac{1}{4}$ in.; in spherical bottoms, never less than that of lower course in cylindrical part of tank. All plates shall be sheared or planed to proper bevel along the edges for calking. All plates shall be calked along beveled edges from inside, with round-nosed tool. Foreign material for calking, such as lead, copper, filings and cement, shall not be used. Plates in

tanks for storage of oil shall be beveled on both sides for outside and inside calking. Radial sections of spherical bottoms shall be made in multiples of number of columns supporting tank, and reinforced at lower parts, where holes are made for piping. When center of spherical bottom is above point of connection with cylindrical part of tank, there shall be a girder at said connection to take horizontal thrust; horizontal girder may be made in connection with balcony. This also applies where tank is supported by inclined columns. Balcony around tank shall be 3 ft. wide, and have floor-plate $\frac{1}{4}$ in. thick, punched for drainage; and shall be provided with a suitable railing, 3 ft. 6 in. high. Upper parts of spherical bottom plates shall always be connected on inside of cylindrical section of tank.

To avoid eccentric loading on tower columns, and local stresses in spherical bottoms, connections between columns and sides of tank shall be made in such manner that arcs through center of gravity of column section intersect center of connection between spherical bottom and sides of tank. Enough rivets shall be provided above this intersection to transmit total column load. If tank is supported on columns riveted directly to sides, additional material shall be provided in tank plates riveted directly to columns to take shear. Shear may be taken by providing thicker tank plates, or by reinforcement plates at column connections, while bending moments shall be taken by upper and lower flange angles. Connections to columns shall be made in such manner that efficiency of tank plates shall not be less than that of vertical seams. For high towers, columns shall have batter of 1 to 12. Hight of tower shall be distance from top of masonry to connection of spherical bottom, or flat bottom, with cylindrical part of tank. Near top of tank there shall be one Z-bar to support painter's trolley, and stiffen tank; its section modulus not less than $\frac{D^2}{250}$, where D is diam. of tank, ft. If upper part of tank is thoroughly held by roof construction, this may be reduced. On larger tanks, circular stiffening angles shall be provided to prevent plates from buckling during wind storms; distance between angles in ft. $= \frac{900}{D}\sqrt{t}$; t = thickness of tank plates, in.

Tank will generally be covered with conical roof of thin plates; pitch 1 on 6. For tanks up to 22 ft. diam., roof plates will be assumed self-supporting. If diam. exceeds 22 ft., angle rafters shall support roof plates, which are generally $\frac{1}{8}$ in. thick. Plates assumed self-supporting: $\frac{3}{32}$-in. plate, up to a diam. of 18 ft.; $\frac{1}{8}$ in., up to 20 ft.; $\frac{3}{16}$-in., up to 22 ft. Rivets in roof plates shall be $\frac{1}{4}$ or $\frac{5}{16}$ in., driven cold; need not be headed with button set.

Trap-door, 2 ft. square, in roof. Platform with railing, for safety of men operating trap-door near top of higher tanks. Ladder 1 ft. 3 in. wide shall extend from about 8 ft. above foundation to top of tank; also one inside of tank; made of two $2\frac{1}{2}$ by $\frac{3}{8}$-in. bars with $\frac{3}{4}$-in. round rungs 1 ft. apart. On high tanks, 30 ft. diam. or greater, walk shall be provided from column nearest ladder to expansion joint on riser pipe. If overflow pipe is not specified, 6 in. shall be added to required hight of tank. Outlet pipe, in most cases, is not required, as inlet pipe will serve.

Pipes entering tank shall have cast-iron expansion joints with rubber packing, and facilities for tightening. Expansion joint, generally, shall be fastened to bottom of tank with bolts having lead washers; tank plates reinforced where pipes enter. Pipes entering tank shall be thoroughly braced laterally with adjustable diagonal bracing at panel points of tower. Diagonal bracing in tower shall preferably be adjustable, and calculated for initial stress of 3000 lbs. in addition to wind stresses, etc. Size and number of anchor-bolts in tower shall be determined

by maximum uplift when tank is empty; where maximum uplift is greater than 10,000 lbs., they shall be fastened directly to columns with bent plates or similar details. In all other cases it will be sufficient to connect anchor-bolts directly to base-plates. Tension in anchor-bolts shall not exceed 15,000 lbs. per sq. in. of net area; minimum section limited to diam. of 1¼ in.; details made so that anchor-bolts will develop full strength; anchor-plate, not less than ½ in. thick, to be sufficient without depending on adhesion between concrete and steel. Concrete foundation, assumed 140 lbs. per cu. ft., shall be sufficient to take uplift, with factor of safety of 1½. Three-ply frost-proof casing, if necessary, around pipes, composed of two layers of ⅞ by 2½-in. dressed lumber, each layer covered with tar paper or tarred felt, and one outside layer of ⅞ by 2½-in. dressed and matched flooring; lumber in lengths of about 12 ft.; 1-in. air space between layers of lumber, and wooden separators nailed every 3 ft. (In very cold climates fill space between pipes and first layer of lumber with hay or similar material.) Casing may be square or cylindrical; braced to tower with adjustable diagonal bracing, as described for pipes above.

Specifications for Steel Standpipes.* Standpipes are economical only where capacity is more important than pressure, or where local conditions are such that elevated tank is not required. In cold climates roofs are generally omitted on account of ice. In warmer climates roofs prevent water from becoming breeding place for mosquitos, flies, etc.

Loads, Stresses. Same as Elevated Steel Tanks, pages 530, 531.

Details of Construction: Sides of standpipe shall be in courses of alternately large and small diam. Joints for horizontal seams in sides and for bottom plates shall preferably be lap joints. Minimum thickness of plates forming sides shall be ¼ in., and $\frac{5}{16}$ in. for bottom. Bottom plates for ordinary standpipes shall be provided with tapped holes, 1¼ in. diam., with screw plugs, spaced at about 4-ft. centers, to permit filling with cement grout on top of foundation while bottom part is being erected, to secure proper bearing. Oil tanks of large diam. are generally set directly on sand foundation, and do not need any holes in bottom plates for grout. In such cases, ¼-in. bottom plates will be sufficient. Bottom plates shall be connected with sides by angle iron riveted inside standpipe. This angle iron shall be bevel-sheared for calking along both legs. On side and near bottom there shall be 12 by 18-in. elliptical manhole. In same manner, or on bottom plates, flanges shall be provided for connection of inlet and outlet pipes. All openings in standpipes shall be properly reinforced by forged rings or plates. Outside ladder, 1 ft. 3 in. wide, extends from about 8 ft. above foundation to top of standpipe; inside ladder will not be required. (In no case should inside ladders be provided in standpipes in climates where ice will form. Owners of oil tanks often specify stairways to take place of ladders.) All ladders to sustain a concentrated load of at least 800 lbs. Standpipes for water shall be set on concrete foundations and anchored thoroughly thereto with anchor-bolts not less than 1¼ in. diam., set deep enough to take necessary uplift, and provided with an anchor-plate not less than ½ in. thick. All anchor-bolts shall be connected directly to sides of standpipe with bent plates or similar details. Unit stress in anchor-bolts shall not exceed 15,000 lbs. per sq. in. of net area.

Specifications for Materials, Workmanship, Inspection, Painting and Testing.* *Materials.* Steel shall be made by open-hearth process. Chemical and physical properties shall conform to following limits:

* G. W. Birch-Nord, Trans Am. Soc. C. E., Vol. 64, 1909.

Table 161. Properties of Steel for Standpipes.

Elements considered	Structural steel	Rivet steel
Phosphorus, maximum { Basic	0.04 %	0.04 %
Phosphorus, maximum { Acid	0.06 %	0.04 %
Sulphur, maximum	0.05 %	0.04 %
Ultimate tensile strength, in lbs. per sq. in.	{ 60,000 desired	{ 50,000 desired
Elongation: minimum per cent. in 8 in. Fig. 304.	1,500,000 / Ult. ten. str.	1,500,000 / Ult. ten. str.
Elongation: minimum per cent. in 2 in. Fig. 305.	22	....
Character of fracture	Silky	Silky
Cold bends without fracture	180° flat	180° flat

Yield point, as indicated by drop of beam, shall be recorded in test reports. If ultimate strength varies more than 4000 lbs. from that desired, a re-test shall be made on same gage, which, to be acceptable, shall be within 5000 lbs. of desired ultimate. Chemical determinations of per cent. of carbon, phosphorus, sulphur, and manganese shall be made by manufacturer from test ingot taken at time of pouring of each melt of steel, and a correct copy of such analysis shall be furnished to engineer. Check analyses shall be made from finished material, if called for, in which case an excess of 25 per cent. above required limits will be allowed. Specimens for tensile and bending tests, for plates, shapes, and bars, shall be made by cutting coupons from finished product, which shall have both faces rolled and both edges milled to form shown by Fig. 304; or with edges parallel; or they may be turned to a diam. of ¾ in. for a length of at least 9 in., with enlarged ends.

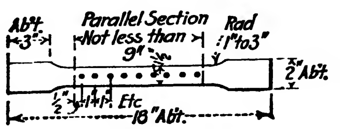


FIG. 304.

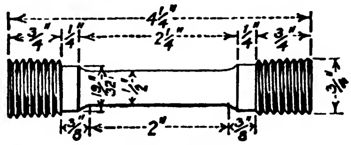


FIG. 305.

Rivet rods shall be tested as rolled. Specimens shall be cut from finished rolled or forged bar, in such manner that center of specimen shall be 1 in. from surface of bar. Specimen for tensile test shall be turned to form shown by Fig. 305; specimen for bending test shall be 1 in. by ½ in. in section. Material to be used without annealing or further treatment shall be tested in condition in which it comes from rolls. When material is to be annealed, or otherwise treated before use, specimens for tensile test representing such material shall be cut from properly annealed or similarly treated short lengths of full section of bar. At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, test shall be made from thickest and thinnest material rolled. For material less than $\frac{5}{16}$ in and more than ¾ in. thick, following modifications will be allowed in requirements for elongation: (*a*) For each $\frac{1}{16}$ in. thickness below $\frac{5}{16}$ in., a deduction of 2½ from specified per cent. will be allowed; (*b*) for each ⅛ in. in thickness above ¾ in., a deduction of 1 from specified per cent. will be allowed. Bending tests may be made by pressure or by blows. Plates, shapes, and bars less than 1 in. thick shall bend as called for in Table 161. Angles ¾ in. and less thick shall open flat, and angles ½ in. and less thick shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by inspector. Rivet steel, when nicked and

bent around bar of same diam. as rivet rod, shall give gradual break and fine, silky, uniform fracture. Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform, workmanlike finish. Plates 36 in. wide and less shall have rolled edges. Every finished piece of steel shall have melt number and name of manufacturer stamped or rolled upon it. Steel for pins shall be stamped on end. Rivet and lattice steel and other small parts may be bundled, with above marks on attached metal tag. Material which, subsequent to foregoing tests at mills, and acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at shop, and shall be replaced by manufacturer at his own cost. Variation in cross-section or weight of each piece of steel of more than 2½ per cent. from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by following permissible variations, which are to apply to single plates:

Plates weighing 12½ lbs. per sq. ft. or more: (*a*) Up to 100 in. wide, 2½ per cent. above or below prescribed weight; (*b*) 100 in. wide or more, 5 per cent. above or below.

Plates weighing less than 12½ lbs. per sq. ft.: (*a*) Up to 75 in. wide, 2½ per cent. above or below; (*b*) 75 in. and up to 100 in. wide, 5 per cent. above or 3 per cent. below; (*c*) 100 in. wide or more, 10 per cent. above or 3 per cent. below.

Plates will be accepted if thickness is not more than 0.01 in. less than ordered. An excess over nominal weight, corresponding to dimensions on order, will be allowed for each plate, 1 cu. in. of rolled steel being assumed to weigh 0.2833 lb., in accordance with Table 162:

Table 162. Steel Plates; Excess Weight Allowed for Payment

Thickness, in	Nominal weight, lbs. per sq ft	Widths of plates		
		Up to 75 in., %	75 in. and up to 100 in., %	100 in. and up to 115 in., %
¼	10.20	10	14	18
5/16	12.75	8	12	16
⅜	15.3	7	10	13
7/16	17.85	6	8	10
½	20.4	5	7	9
9/16	22.95	4½	6½	8½
⅝	25.5	4	6	8
More than ⅝	.	3½	5	6½

Cast Iron. Except where chilled iron is specified, castings shall be made of tough, gray iron, with not more than 0.10 per cent. sulphur. They shall be true to patterns, out of wind, and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on "Arbitration Bar" of Am. Soc. for Testing Materials, which is a round bar, 1¼ in. diam. 15 in. long. Transverse test shall be made on supported length of 12 in. with load at middle. Minimum breaking load thus applied shall be 2900 lbs., with deflection of at least 1/10 in. before rupture.

Workmanship, Inspection. All material shall be throughly straightened in shop, by methods which will not injure it, before being laid off or worked in any way. Shearing shall be done neatly and accurately, and all portions of work exposed to view shall have neat and uniform appearance. Size of each rivet, called for by plans, shall be understood to mean actual size of cold rivet before heating. All plates and shapes shall be shaped to proper curve by cold rolling; heating or hammering for straightening or curving will not be allowed. Plates to be scarfed may be heated to cherry-red color, but not hot enough to ignite a

piece of dry wood when applied to it. Most careful attention shall be paid to all scarfing. All plates or shapes shall be punched before being bevel sheared or planed for calking. All screw threads shall make tight fits in nuts and turnbuckles, and shall be U. S. Standard, except for diam. greater than 1⅜ in., when they shall have six threads per in. Dimensions of screws of various sizes shall be as follows:

Diam. of screw ends	1 in.	1⅛ in.	1¼ in.	1⅜ and greater
Number of threads per inch.	8	7	7	6

Minimum excess of upset bar at root of thread over body of bar shall be 15 per cent. Shape of thread shall be U. S. Standard.

Table 163. Standard Upsets for Round and Square Steel Bars

Round bars		Square bars	
Bar	Upset	Bar	Upset
Diam., in.	Diam., in	Side, in.	Diam., in.
¾	1	¾	1⅛
⅞	1⅛	⅞	1¼
1	1⅜	1	1½
1⅛	1½	1⅛	1⅝
1¼	1⅝	1¼	1⅞
1⅜	1¾	1⅜	2
1½	1⅞	1½	2¼
1⅝	2	1⅝	2⅜
1¾	2⅛	1¾	2½
1⅞	2¼	1⅞	2¾
2	2⅜	2	2⅞

Diam. of die used in punching rivet holes shall not exceed that of punch by more than $\frac{1}{16}$ in. All rivet holes shall be punched, except as stated previously. All punched and reamed holes shall be clean cuts, without torn or ragged edges. Burrs on all reamed holes shall be removed by tool, countersinking not more than $\frac{1}{16}$ in. Any parts of structure in which difficulties may arise in field riveting shall be assembled in shop and marked properly before shipment. Rivet holes shall be accurately spaced; eccentrically located rivet holes, if not sufficient to cause rejection, shall be corrected by reaming, and rivets of larger size shall be used in holes thus reamed. Use of drift-pins will be allowed only for bringing together several parts; force will not be allowed in drifting under any circumstances. Use of sledges on any part will not be allowed. Care shall be taken to prevent material from falling, or being in any way subjected to heavy shocks. Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand-driving. All rivet heads shall be concentric with holes. All calking shall be done with round-nosed tool, and only by skilled men. Calking around rivet heads will not be allowed. All fractured material shall be replaced free of cost to owner.

Painting and Testing. Before leaving shop, all steelwork except laps in contact on tank work shall receive one coat of approved paint or boiled linseed oil. All parts which will be inaccessible after erection shall be well painted, except as just stated. After structure is erected and all seams calked, it shall be tested for water-tightness, and leaky places shall be marked. Water shall then be discharged and leaky seams calked; leaky rivets shall be cut out and replaced with new ones. After structure has stood empty 3 days, it shall be retested, and then, if all joints are water-tight, it shall be given one coat of approved paint both inside and outside. Painting in open air shall never be done in wet or freezing weather. Contractor

shall guarantee tightness of tank, or standpipe, when filled with liquid it is designed to contain.

Foundations for Tanks on Towers, and for Standpipes.* Average permissible pressure on soil, tons per sq. ft.: Soft clay, 1; ordinary clay, 2; dry sand and dry clay, 3; hard clay, 4; gravel and coarse sand, 6. Foundations shall be carried below frost, and anchor-bolts placed deep enough to develop full strength. In foundations for towers with inclined legs supporting elevated tanks, care shall be taken that piers are constructed in such manner that resultant of vertical and horizontal forces, due to direct loads, passes through center of gravity of piers. Foundations, in general, shall be concrete of 1 part Portland cement, 3 parts sand, and 5 parts crushed stone or gravel; where part of foundation is under water, concrete shall be 1:2:4 mixture. On earth or other yielding foundation, if a standpipe is enclosed by a masonry tower, the two foundations should be separated by a joint permitting independent movement. This is especially desirable because of change of load as the standpipe is filled and emptied.

Overflow. In localities where water in a standpipe may freeze, the overflow should be outside, connected by an aperture through the plate at the top.

Experience. *Inside calking* on standpipes having large and small rings requires every second horizontal joint to be calked upward. Some engineers consider this method unsatisfactory, owing to inconvenient position of calker, generally resulting in poor work. By using tapered rings, with smaller diam. on top inside, downward calking is convenient. All riveted joints should be calked from side from which rivet is driven. Tail of a driven rivet generally shows small cracks which may cause a leak under pressure, and calking joint on pressure side would not prevent it. In using tapered rings, rivets are driven from inside and seams are calked from inside. Some have found it unsatisfactory to calk both inside and outside of joint, as one operation is likely to undo all good of other.

Necessity for anchorage can be determined from following equations:

$$W \mathrel{\overset{=}{<}} pkH^2,$$

in which W = total weight of metal in standpipe, lbs.; p = intensity of wind pressure on normal surface, lbs. per sq. ft.; k is reduction factor, to be used on account of curvature of surface; H = total hight of standpipe, ft. This formula can be easily derived, on assumption that anchorage is needed when tension on windward side, due to overturning effect of wind, is equal to compression due to weight of pipe. When anchor-bolts are required, they should be spaced equally around circumference. Stress, lbs., in an anchor-bolt may be determined from equation:

$$S = \left(\frac{W - pkH^2}{D'}\right)\frac{1}{n},$$

in which D' = diam. of anchor-bolt circle, ft.; n = number of equally-spaced anchor-bolts. This equation is readily obtained by considering standpipe as cantilever supported by anchor-bolts.

For connection of shell with flat bottoms, use angle irons having thickness

* G. W. Birch-Nord, Trans. Am. Soc. C. E., Vol. 64, 1909.

Diam. ft.	Outside circumference of inner course, ft.	Bottom or cross-section, area, sq ft	Hight, ft	Capacity, million gallons	Required thickness of lowest plate, unit stress not exceeding 10,000 lbs., in *	Thickness of bottom, in	Approximate weight of steel, tons	Weight on foundation, tank full: Total tons	Weight on foundation, tank full: Tons per sq ft.	Rivet data, vertical seams (a): Diam., in.	Rivet data (a): Pitch, in.	Rivet data (a): Distance between gage lines, in.	Rivet data (a): Type of joint and assumed efficiency, %	Rivet data (a): Distance of gage line from edge, in.	Paint, 2 field coats (b): Circumferential area, sq. ft.	Paint (b): Gals. of graphite at 350 sq. ft. per gal.	Paint (b): Gals. of asphaltum at 310 sq. ft. per gal
20	62 96	314	20	0.047	[illegible]	[illegible]	9	205	0 65	[illegible]	1 [illegible]		Single lap 50	1	1260	4	4
20	''	''	30	0 071	[illegible]	[illegible]	12	307	0 98	[illegible]	1 [illegible]		Single lap 50	1	1890	5	5
20	''	''	40	0 094	[illegible]	[illegible]	16	409	1 30	[illegible]	1 [illegible]		Single lap 50	1	2520	7	8
20	63 00	''	50	0.118	[illegible]	[illegible]	20	511	1.63	[illegible]	1 [illegible]	.....	Single lap 50	1 [illegible]	3150	9	10
25	78 67	491	20	0.074	[illegible]	[illegible]	12	319	0 65	[illegible]	1 [illegible]	.....	Single lap 50	1	1570	4	5
25	''	''	30	0.110	[illegible]	[illegible]	16	476	0 97	[illegible]	1 [illegible]	...	Single lap 50	1	2360	6	7
25	78 70	''	40	0.147	[illegible]	[illegible]	21	635	1 29	[illegible]	1 [illegible]	. .	Single lap 50	1 [illegible]	3150	9	10
25	78 74	''	50	0 184	[illegible]	[illegible]	26	793	1 62	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	3830	11	12
25	78 77	''	60	0 221	[illegible]	[illegible]	34	954	1 94	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	4720	13	15
25	78.80	''	70	0 258	[illegible]	[illegible]	42	1116	2 27	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	5420	15	17
30	94 38	707	20	0 105	[illegible]	[illegible]	15	457	0 65	[illegible]	1 [illegible]	.. .	Single lap 50	1	1890	5	6
30	''	''	30	0 158	[illegible]	[illegible]	20	683	0 97	[illegible]	1 [illegible]	. .	Single lap 50	1	2930	8	10
30	94 41	''	40	0 211	[illegible]	[illegible]	26	910	1 29	[illegible]	1 [illegible]	. .	Single lap 50	1 [illegible]	3770	11	12
30	94 47	''	50	0 264	[illegible]	[illegible]	37	1142	1 62	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	4720	13	15
30	94 51	''	60	0 316	[illegible]	[illegible]	47	1372	1.94	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	5670	16	18
30	94 54	''	70	0 369	[illegible]	[illegible]	59	1605	2 27	[illegible]	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	6620	19	21
30	94 58	''	80	0 422	[illegible]	[illegible]	72	1839	2 60	[illegible]	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	7570	22	24
30	94 64	''	90	0 475	[illegible]	[illegible]	89	2077	2 94	1	3	2 [illegible]	Double butt 70	1 [illegible]	8520	24	28
30	94 67	''	100	0 528	[illegible]	[illegible]	105	2314	3 27	1	3	2 [illegible]	Double butt 70	1 [illegible]	9460	27	31
30	94 71	''	110	0 580	[illegible]	[illegible]	124	2554	3 61	1 [illegible]	3	2 [illegible]	Double butt 70	2	10410	30	34
30	94 74	''	120	0 633	[illegible]	[illegible]	144	2795	3 95	1 [illegible]	3	2 [illegible]	Double butt 70	2	11360	32	37
35	110 08	962	20	0 144	[illegible]	[illegible]	18	619	0 64	[illegible]	1 [illegible]	......	Single lap 50	1	2200	6	7
35	110 12	''	30	0 215	[illegible]	[illegible]	25	927	0 96	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	3310	9	11
35	110 15	''	40	0 287	[illegible]	[illegible]	37	1240	1 29	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	4410	13	14
35	110 22	''	50	0 359	[illegible]	[illegible]	48	1551	1 61	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	5510	16	18
35	110 25	''	60	0.431	[illegible]	[illegible]	61	1865	1 94	[illegible]	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	6610	19	21
35	110 32	''	70	0.502	[illegible]	[illegible]	80	2185	2 27	1	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	7710	22	25
35	110 35	''	80	0 574	[illegible]	[illegible]	98	2503	2 60	1	3	2 [illegible]	Double butt 70	1 [illegible]	8830	25	28
35	110 42	''	90	0 646	[illegible]	[illegible]	118	2824	2 94	1 [illegible]	3	2 [illegible]	Double butt 70	2	9930	28	32
35	110 45	''	100	0 718	[illegible]	[illegible]	141	3148	3 27	1 [illegible]	3	2 [illegible]	Double butt 70	2	11040	31	35
35	110 51	''	110	0 790	1 [illegible]	[illegible]	166	3473	3 61	—	—	—	—	—	12140	35	39
35	110 54	''	120	0 862	1 [illegible]	[illegible]	195	3803	3 95	—	—	—	—	—	13240	38	43
35	110 57	''	130	0 934	1 [illegible]	[illegible]	225	4134	4 30	—	—	—	—	—	14350	41	46
40	125 79	1257	20	0 188	[illegible]	[illegible]	21	807	0 64	[illegible]	1 [illegible]	......	Single lap 50	1	2520	7	8
40	125 82	''	30	0 282	[illegible]	[illegible]	30	1208	0 96	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	3780	11	12
40	125 89	''	40	0 376	[illegible]	[illegible]	45	1616	1 29	[illegible]	2 [illegible]	2 [illegible]	Double lap 60	1 [illegible]	5090	14	16
40	125 95	''	50	0 470	[illegible]	[illegible]	60	2024	1 61	[illegible]	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	6300	18	20
40	125 98	''	60	0 564	[illegible]	[illegible]	77	2434	1.94	[illegible]	2 [illegible]	2 [illegible]	Double butt 70	1 [illegible]	7560	22	24
40	126 05	''	70	0 658	[illegible]	[illegible]	101	2851	2 27	1	3	2 [illegible]	Double butt 70	1 [illegible]	8820	25	28
40	126 11	''	80	0.751	[illegible]	[illegible]	125	3268	2 60	1 [illegible]	3	2 [illegible]	Double butt 70	2	10080	29	33
40	126 14	''	90	0 846	[illegible]	[illegible]	151	3686	2 93	1 [illegible]	3	2 [illegible]	Double butt 70	2	11350	32	36
40	126 21	''	100	0 941	1 [illegible]	[illegible]	184	4112	3 27	—	—	—	—	—	12620	36	41
40	126 24	''	110	1 035	1 [illegible]	[illegible]	216	4537	3 61	—	—	—	—	—	14080	40	45
40	126 31	''	120	1 130	1 [illegible]	[illegible]	251	4965	3 95	—	—	—	—	—	15130	43	49

* Thin cylinder formula, [illegible] in is thinnest plate to be considered in "Steel Mill Buildings," from Pencoyd Handbook (a) Johnson's "Framed Structures" based on Watertown tests. (b) Quoted by Ketchum

equal, at least, to that of bottom ring of shell. In all cases where triple riveting is necessary for lower horizontal joints, two bottom angles are used, in which case thickness of each angle is not less than two-thirds of thickness of bottom ring.

Materials, Life and Arrangement of Towers. A water tower may have three or more legs. With wooden towers it is desirable to use 12 and even more legs, in case the tank is very large, because a lesser number of 12 by 12's, the size of timber most suitable, would not give sufficient cross-section. Most builders have now adopted the 4-post steel design, as being, everything considered, the best, except for tanks of very large capacities.

Form of Tank Bottom. When steel was first substituted for wood, flat bottoms were used. This was uneconomical owing to the heavy floor system required, and the inaccessibility of the bottom for painting. A conical form has in some instances been used, but there is nothing to recommend this type, while a few notable failures are recorded against it. The most practical and economical form for the bottom of steel tanks is the hemisphere. Shops are equipped with special machinery for the manufacture of these tanks and carry a stock of material for certain of the standard sizes for quick delivery.

Table 165. Dimensions of Standard Steel Water Tanks*

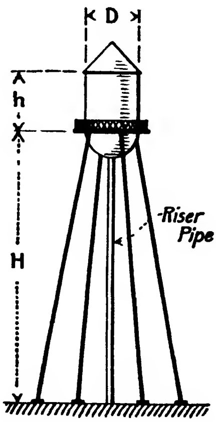


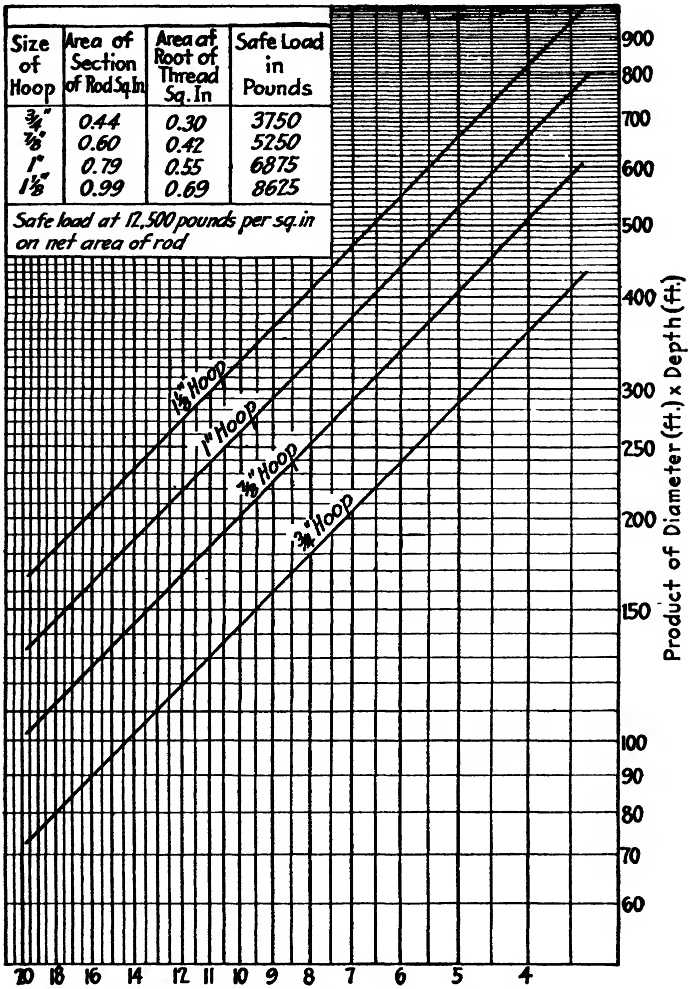
Fig. 282.

Capacity, gals	Diam., ft., D	Height, h ft	in
10,000	11	10	0
15,000	12	14	0
20,000	13	16	0
25,000	14	17	0
30,000	15	18	0
35,000	16	18	0
40,000	17	18	0
45,000	17	21	0
50,000	18	20	4
55,000	18	23	0
60,000	19	22	0
65,000	19	24	4
70,000	20	23	0
75,000	20	25	3
80,000	21	24	0
90,000	21	27	9
100,000	22	28	0
125,000	24	29	0
150,000	25	32	6
175,000	26	35	5
200,000	28	34	1
250,000	30	37	4
300,000	32	39	3

* Des Moines Bridge and Iron Co. Company has drawings and templates and carries stock of materials for all size up to and including 100,000 gals capacity. Presumably other large companies are similarly prepared for quick delivery. Rule for finding distances, center to center of foundations, at tops of cap stones:
Square $= 0.707D + 0.2H + 1\frac{3}{4}$ in. Diagonal $= D + 0.2828H + 2\frac{1}{4}$ in. For capacities of cylindrical tanks, see Table 213, page 630.

WOODEN TANKS*

Comparison of Wooden and Steel Tanks. Steel tanks of sizes commonly used for fire protection cost from 40 to 100 per cent. more than wooden; the additional cost of large tanks is relatively less. A steel tank of about 40,000 gals. capacity, or over, can be erected on a steel trestle at about the same cost as a wooden tank, since a saving can be made in the cost of supports by a hemispherical bottom. A steel tank is superior to a wooden: (1) It will last



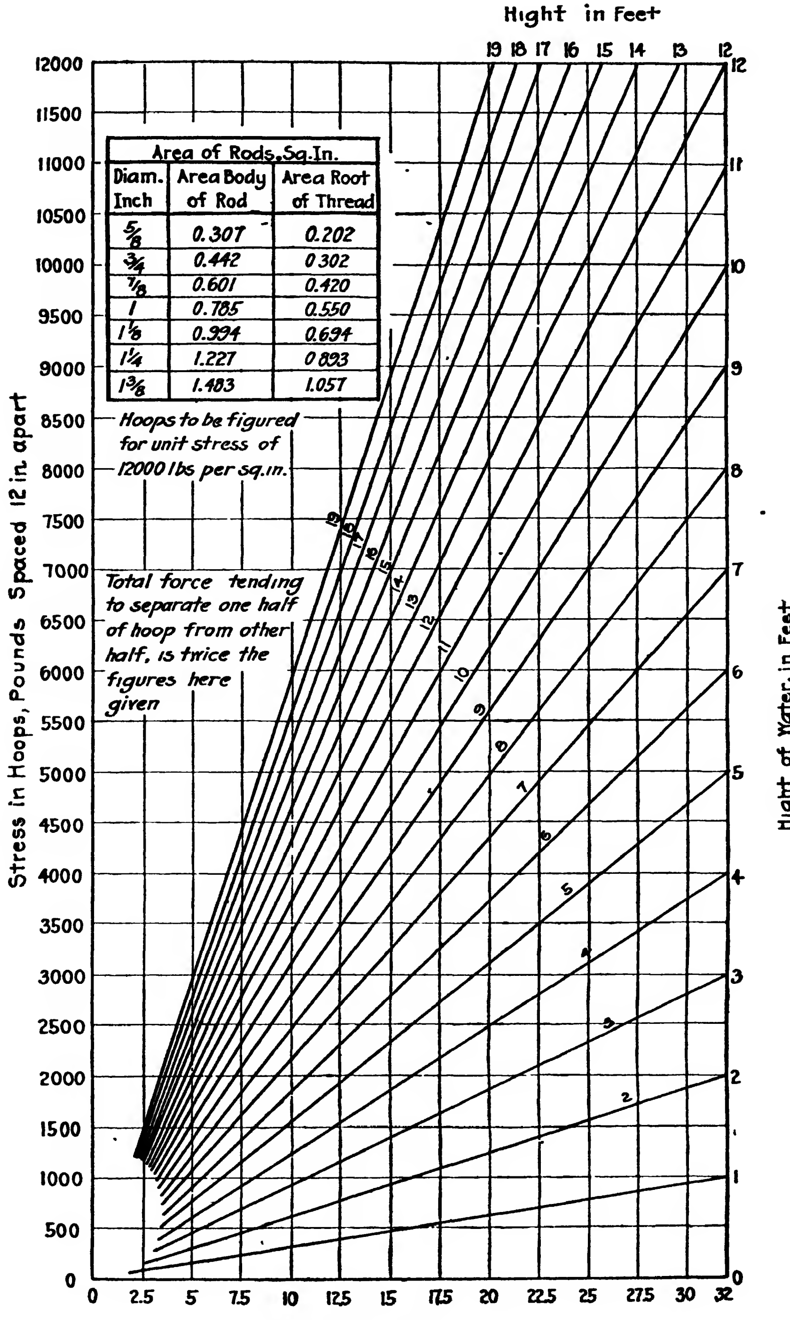
Size of Hoop	Area of Section of Rod Sq. In.	Area at Root of Thread Sq. In.	Safe Load in Pounds
¾"	0.44	0.30	3750
⅞"	0.60	0.42	5250
1"	0.79	0.55	6875
1⅛"	0.99	0.69	8625
Safe load at 12,500 pounds per sq. in on net area of rod			

$$\text{Spacing, inches} = \frac{\text{safe load per hoop, lbs.}}{2.6 \times \text{diam. ft.} \times \text{depth, ft.}}$$

FIG. 306.—Allowable spacing of hoops† on wooden tanks.

* Lack of space prevents a full presentation of especially valuable specifications prepared by Inspection Dept., Assoc. Factory Mutual Fire Insurance Cos. 1903. Tanks of less than 10,000 gals. capacity not included.

† Inspection Dept., Assoc. Factory Mutual Fire Ins. Co.



Area of Rods, Sq. In.

Diam. Inch	Area Body of Rod	Area Root of Thread
5/8	0.307	0.202
3/4	0.442	0.302
7/8	0.601	0.420
1	0.785	0.550
1 1/8	0.994	0.694
1 1/4	1.227	0.893
1 3/8	1.483	1.057

Fig. 307.—Stress in hoops of cylindrical wooden tanks for varying depths of water and diameters of tank.

(Ballinger and Perrot, 1903.)

Table 166. Capacities and Weights of Small Wooden Water Tanks*

Regular sizes from 1½- and 2-in. lumber

(U. S. Wind Engine and Pump Co)

Length of stave, ft.	Diam. of bottom, ft.	No. hoops	Capacity, bbls.	Capacity, gals.	Approximate weight, 2-in. pine, or 1½-in. cypress, lbs.	Length of stave, ft.	Diam. of bottom, ft.	No. hoops	Capacity, bbls.	Capacity, gals	Approximate weight, 2-in. pine, or 1½-in. cypress, lbs.
2	4	2	4	117	180	2	8	2	17	543	450
2.5	4	2	5	158	210	2.5	8	2	23	710	510
4	4	4	8	268	325	3	8	3	28	884	590
5	4	4	11	342	385	4	8	4	39	1,218	740
6	4	5	13	410	460	6	8	5	60	1,878	1,000
2	5	2	6	195	240	7	8	6	70	2,207	1,170
2.5	5	2	8	255	280	8	8	7	80	2,535	1,345
4	5	4	14	443	425	10	8	8	101	3,180	1,520
5	5	4	18	562	500	12	8	9	121	3,816	1,700
6	5	5	22	675	590						
7	5	6	25	784	690	2	9	2	22	696	645
						2.5	9	2	29	904	710
2	6	2	9	292	290	6	9	5	76	2,390	1,250
2.5	6	2	12	382	330	8	9	7	102	3,210	1,550
3	6	3	15	477	400	10	9	8	128	4,020	1,830
4	6	4	21	659	480	12	9	9	153	4,820	2,125
5	6	4	27	838	550						
						2	10	2	28	870	750
6	6	5	32	1,017	640	2.5	10	2	36	1,138	825
7	6	6	38	1,190	740	8	10	7	129	4,060	1,865
8	6	7	43	1,368	850	10	10	8	162	5,102	2,150
10	6	8	54	1,710	960	12	10	9	181	5,700	2,465
2	7	2	13	408	360	2	12	2	40	1,270	1,000
2.5	7	2	17	534	410	2.5	12	3	53	1,661	1,116
6	7	5	45	1,413	900	8	12	7	189	5,944	2,072
7	7	6	53	1,660	1,025	10	12	8	237	7,476	2,421
8	7	7	60	1,886	1,150	12	12	9	286	8,998	2,770
10	7	8	74	2,335	1,300	14	12	12	335	10,544	3,200

The average life of wooden tanks is about 15 yrs. * See also Table 213, p. 630.

for an indefinite time if kept thoroughly painted inside and out; whereas a wooden tank will have to be replaced in from 12 to 30 yrs. (usually 15). (2) It will be absolutely tight when once well erected and properly cared for, whereas a wooden tank will shrink and leak when the water gets low. (3) It will not be likely to burst suddenly (if originally correctly designed) even if painting is neglected, for a few spots will rust through first. The objections to steel tanks are: (1) They require skilled boiler makers to erect, thus adding considerably to the cost when at a distance from the boiler shop. (2) They are more difficult to protect against freezing. (3) They give more trouble by sweating when in a building. (4) They deteriorate rapidly if painting is neglected. (Inspection Dept., Assoc. Factory Mutual Fire Ins. Cos.)

Specifications for Cylindrical Wooden Tanks.† *Material.* Cedar, cypress white pine, or Oregon pine (Douglas fir), free from sap, loose or unsound knots, worm holes and shakes; thoroughly air dired. Staves and bottom of 2½-in. (dressed to 2¼-in) stock, for tanks not exceeding 16 ft. diam., or 16 ft. deep; for larger tanks, 3 in., dressed to about 2¾ in. Tanks of the best red cedar and of good

† Inspection Dept., Assoc. Factory Mutual Fire Ins. Cos.

Table 167. Capacities and Weights of Standard Size Wooden Railroad and Storage Tanks

Made from 3-in. material

(U. S. Wind Engine and Pump Co.)

Length stave, ft.	Diam., ft.	No. hoops	Capacity		Weight, lbs.	
			Gals.	Bbls	Pine	Cypress
10	10	8	4,750	150	3,100	3,700
12	10	9	5,700	182	3,500	4,300
14	10	10	6,680	212	4,000	4,800
10	12	8	7,053	224	4,000	4,800
12	12	9	8,488	269	4,500	5,600
14	12	10	9,902	314	5,200	6,300
16	12	12	11,293	358	5,800	7,100
10	14	8	9,773	310	4,800	5,900
12	14	9	11,774	374	5,500	6,700
14	14	10	13,750	436	6,200	7,600
16	14	12	15,701	498	6,900	8,500
10	16	8	12,935	410	5,700	7,000
12	16	9	15,597	495	6,700	8,200
14	16	10	18,229	579	7,300	9,000
16	16	12	20,833	661	8,200	10,000
18	16	14	23,406	743	9,200	11,200
12	18	9	19,956	633	7,700	9,300
14	18	10	23,340	741	8,700	10,600
16	18	12	26,689	847	9,700	11,800
18	18	14	30,004	952	10,900	13,200
20	18	15	33,288	1,057	11,900	14,400
12	20	9	24,852	788	9,000	10,100
14	20	10	29,080	923	10,100	12,200
16	20	12	33,270	1,056	11,200	13,600
18	20	14	37,423	1,191	12,500	15,100
20	20	15	41,540	1,319	13,700	16,500
12	22	10	30,285	961	10,300	12,500
14	22	11	35,451	1,125	11,500	13,900
16	22	12	40,576	1,288	12,600	15,300
18	22	14	45,660	1,449	14,000	17,000
20	22	16	50,702	1,609	15,400	18,600
12	24	10	36,254	1,151	11,600	14,000
14	24	11	42,453	1,347	12,900	15,600
16	24	13	48,606	1,543	14,300	17,300
18	24	14	54,714	1,737	15,700	19,000
20	24	16	60,778	1,929	17,300	20,800
14	26	11	50,071	1,589	13,800	18,100
16	26	13	57,360	1,821	15,700	19,000
18	26	14	64,587	2,050	17,200	20,800
20	26	16	71,766	2,278	19,000	22,900
16	28	14	66,785	2,121	17,500	22,700
18	28	15	75,449	2,395	18,800	24,400
14	30	12	67,150	2,132	17,300	20,800
16	30	14	77,044	2,446	19,900	23,900
18	30	15	86,790	2,755	21,500	25,800
20	30	17	96,480	3,063	23,700	28,300

cypress are about equally durable. For use out of doors, or where exposed to freezing, Michigan pine, free from sap wood, is generally the most durable; ordinary Eastern pine will not last as long as any of the woods above mentioned. Cypress or cedar tanks will last at least 15 yrs. and commonly 20 or 25. One cedar tank is reported in good condition after 42 yrs.' service.

Dressing of Lumber. Staves to be sawed to the proper bevel to make true joints and correct taper; edges may or may not be planed. If not planed, care should be exercised to have a very carefully sawn surface. If planed, the planing should be done in the very best manner, because any little wavings may cause small openings directly through. The croze (groove for receiving the bottom) to be cut in a true line at a uniform distance from the bottom of the staves. A hole to be bored in both edges of each stave, about one-third the distance from the top, for a ⅜-in. dowel, to hold the staves in position while the tank is being erected. Small metal dogs driven into the tops of the staves may be used instead. Bottom planks are to be dressed on 4 sides, and the edges of each bored with holes not over 3 ft. apart for ⅜-in. dowels.

Hoops. Round wrought iron or mild steel of good quality. Wrought iron is preferable as it does not rust so easily. Wrought iron to have a tensile strength of at least 50,000 lbs. per sq. in.; steel, not less than 55,000, nor more than 60,000; carbon not to exceed 0.10 per cent. Use no welds in any hoops. Where more than one length of iron is necessary, lugs are to be used to make the joints; and when more than one piece is necessary, the several pieces constituting one hoop should be tied together in preparing for shipment. Hoops to be of such size and spacing that the stress will not exceed 12,500 lbs. per sq. in., computed from the area at root of thread. On account of the swelling of the bottom planks, hoops near the bottom may be subjected to a stress greater than that due to the water pressure alone; therefore, additional hoops should be provided. For tanks up to 20 ft. diam., one hoop of the size used next above it should be placed around the bottom opposite the croze, and not counted upon as withstanding any water pressure; for tanks 20 ft. or more in diam., 2 hoops. Hoops with upset ends are not allowed, because: (1)the metal is likely to be burned in upsetting unless done under careful supervision; (2) the additional diam. is likely to be gained by welding bolt ends to smaller rods, thus introducing a weak place at the welds. When the screw threads are cut directly on the ends of a hoop, the unthreaded portions may be corroded to a depth equal to the thread depth without weakening the hoop. The screw thread itself is not so liable to serious corrosion as is the portion of the hoop which bears against the staves, since the latter is subjected to moisture from the wood. No space between hoops to be greater than 21 in. The top hoop to be placed within 2 in. of the top of the staves in order that the overflow pipe may be inserted as high as possible. Hoops to be placed so that the lugs will not come in a vertical line. No hoop to be less than ¾ in. diam. All hoops are to be thoroughly cleaned of mill scale and rust and given one coat of red lead, lamp black and boiled oil, before erecting; and after erecting, another coat of some durable oil or asphaltum paint. Flat hoops, especially those of steel, rust from the back where they bear against the staves, and serious accidents have happened by hoops bursting on account of this unobserved corrosion. Sometimes galvanizing or painting is depended upon to prevent rusting, but there is always the possibility that spots of the protective coating will be knocked off during handling, and one spot left unprotected may result in failure. Round hoops are advised for all tanks.

CHAPTER XXIV

WATER CONSUMPTION

Per Capita Consumption in U. S. cities and towns ranges approximately from 50 to 400 g. p. d. For communities having service connections wholly or largely metered, it is commonly under 100 g. p. d.; and for small cities and towns often much less. For large cities with few meters, but well managed works in good condition, 125 to 150 g.p.d. is a reasonable allowance. Character of industries, climate and other local conditions have important influences.

Lawns, Closets and Bathrooms. Water for sprinkling 100 sq. ft. of lawn, about 1 cu. ft., or 7 to 8 gals.; for soaking 100 sq. ft. of lawn, about 2½ cu. ft., or from 15 to 16 gals.; to flush closet each time, 5 to 6 gals.; to fill lavatory, ordinary, about 1½ gals.; to fill bath tub, ordinary, about 30 gals.

Farm Animals. Consumption of water by farm animals varies greatly, depending upon season of year, age and individual habits of animal, and local conditions. Following is an approximation: Horses, each, 5 to 10 gals. per day; cattle, each, 7 to 12 gals. per day; hogs, each, 1½ to 2½ gals. per day; sheep, each, 1 to 2 gals. per day.

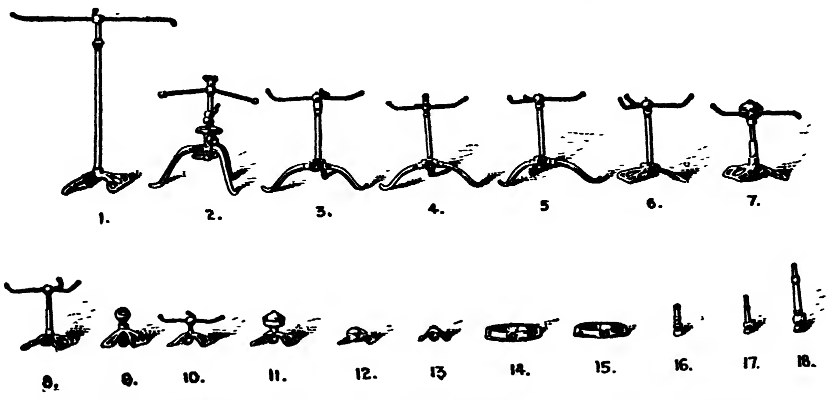


FIG. 308.—Types of sprays and nozzles tested for quantities discharged, Table 170, page 547.
(Jl. N. E. W. W. Ass'n, Dec., 1914.)

Water Required for Steam Making. Standard fixed by Watt for determining horsepower of boilers, was 1 cu. ft. of water evaporated per hour from 212° F. for each horsepower. At that time this was the requirement of the best engines. Prof. Thurston estimated water required per hour per horsepower for good engines equal to 200 divided by square root of pressure, and for best engines, as low as 150 divided by square root of pressure. This would give, for good engines, working with 64 lbs. per sq. in. steam pressure, evaporation of 25 lbs. water, and for best engines, working with 100 lbs. pressure, only 15 lbs. per hr. per hp. = 72 and 43 g.p.d., respectively.

Table 168. Acres Irrigated with Various Rates of Pumping

Diameter of discharge, in.	Gal.	Cu. ft		Number of acres covered in 12 hrs. pumping							
	Per min.	Per sec.	Per min	1 in. deep	2 in deep	3 in. deep	4 in. deep	6 in. deep	8 in. deep	10 in. deep	12 in. deep
1	20	0 045	2.70	0.37	0.183	0.122	0.0915	0.061	0.0457	0.0366	0.0305
1½	50	0.11	6.60	0.89	0.445	0.293	0.2225	0.148	0.1100	0.0890	0.0740
2	100	0.22	13.20	1.80	0.900	0.600	0.4500	0.300	0.2250	0.1800	0.1500
2½	150	0.33	19.80	2.70	1.350	0.900	0.6750	0.450	0.3370	0.2700	0.2250
3	225	0.50	30.00	4.08	2.040	1.360	1.0200	0.680	0.5100	0.4080	0.3400
3½	300	0.66	39.60	5.39	2.695	1.790	1.3500	0.898	0.6730	0.5390	0.4400
4	400	0.90	54.00	7.35	3.675	2.450	1.8400	1.225	0.9200	0.7350	0.6100
5	700	1.60	96.00	13.00	6.500	4.300	3.2500	2.160	1.6200	1.3000	1.0800
6	900	2.00	120.00	16.40	8.200	5.460	4.1000	2.730	2.5000	1.6400	1.3600
7	1,200	2.70	162.00	22.08	11.040	7.360	5.5200	3.680	2.7600	2.2080	1.8400
8	1,600	3.50	210.00	28.60	14.300	9.500	7.1500	4.760	3.5700	2.8600	2.3800
10	3,000	6.70	402.00	54.70	27.350	18.200	13.6750	9.110	6.8300	5.4700	4.5500
12	4,500	10.00	600.00	81.60	40.800	27.200	20.4000	13.600	10.2000	8.1600	6.8000
14	6,000	13.30	800.00	109.00	54.500	36 300	27.2500	18.100	13.6000	10.9000	9.0800
15	7,000	15.50	930.00	126.50	63.250	42.130	31.6250	21.800	15.8000	12.6500	10.5000
16	8,500	18.90	1134.00	143.00	71.500	47.300	35.7500	23.800	17.9000	14.3000	11.9000
18	10,000	22.20	1332.00	181.30	90.650	60.400	45.3200	30.210	22.6000	18.1300	15.1000
20	14,000	31.10	1866.00	254.00	127.000	84.000	63.5000	42.400	31.7000	25.4000	21.1600
22	17,000	37.80	2268.00	308.70	154.350	102.900	77.1700	51.450	38.6000	30.8700	25.7200
24	23,000	51.10	3066.00	417.30	208.650	139.100	104.3200	69.160	52.1600	41.7300	34.7700
26	30,000	66.70	4000.00	544.40	272.200	181.400	136.1000	90.730	68.5000	54.4400	45.3600
30	35,000	77.80	4600.00	634.20	317.100	211.400	158.5500	105.700	78.2500	63.4200	52.8500
36	50,000	111.10	6666.00	907.20	453.600	302.400	226.8000	151.200	113.4000	90.7200	75.6000

Water for Irrigating.* Making allowance for evaporation it requires 28,320 gals. to irrigate 1 acre 1 in. deep. It requires from 10 in. to 20 in. per acre to produce a crop, average being 16 in. Actual amount required depends upon crop and season. Flooding alfalfa 6 in. deep will wet the soil 4 ft. from surface. Flooding orchards 4 in. deep will wet the soil 4 ft. from surface.

Table 169. Daily Consumption of Water by Crops
(Risler)

	Inches		Inches
Lucern............	0.134 to 0.267	Wheat..... . ..	0.106 to 0.110
Meadow grass.....	0.122 to 0.287	Rye............ ..	0.091
Oats....	0.140 to 0.193	Potatoes..	0.038 to 0.055
Corn..	0.111 to 1.570	Oak trees	0.038 to 0.050
Clover..	0.140	Fir trees..	0.020 to 0.043
Vineyard	0.035 to 0.031		

Table 170. Discharges from Lawn Sprinklers† and Garden Hose Nozzles

No. (see Fig 308)	Name and type	Diameters, in. Inlet	Diameters, in. Outlet	No of outlets	Pressure, lbs per sq. in At sillcock	Pressure, lbs per sq. in At nozzle	Length of ¾-in. hose, ft.	Discharge, gals per hr.	Value of water per hr. at 10 cts. per 1000 gals.
1	E. S. Hotchkiss, whirl.	0.60	0 062	15	52	51	50	345	$0 034
					51	48	100	330	0 033
					28	25	100	255	0 025
					25	23	50	225	0.022
2	Double Comet, whirl and jet	0 62	0.142	1	57	51	50	390	0 039
			0 080	3	30	25	50	360	0 036
			0 036	49	57	50	100	330	0 033
					33	25	100	240	0 024
3	Crescent, whirl .	0.62	0 062	15	48	25	100	600	0.060
			0.072	15	43	24	100	540	0 054
					49	34	50	570	0.057
					41	25	50	510	0 051
4	No name, whirl.	0.61	0.070	15	53	41	50	480	0 048
					36	25	50	390	0 039
					54	42	100	420	0 042
					33	25	100	300	0 030
5	No name, whirl	0.60	0.062	15	55	42	100	450	0 045
			0.055	2	35	25	100	330	0 033
					53	47	50	450	0 045
					34	25	50	330	0 033
6	No name, whirl..	0.60	0.061	11	56	49	50	390	0 039
			0 055	2	30	25	50	240	0 024
					55	44	100	360	0.036
					35	25	100	240	0.024
7	Hotchkiss, whirl	0.41	0.041	52	50	30	100	540	0 054
			0 056	15	43	25	100	480	0 048
					48	44	50	570	0 057
					37	25	50	480	0 048
8	Hotchkiss, whirl.	0.41	0.056	15	55	47	50	390	0.039
					32	25	50	300	0 030
					55	46	100	330	0.033
					36	25	100	240	0 024
9	Hotchkiss, whirl.	0.52	0.040	60	46	31	50	660	0.066
					40	25	-----	600	0.060
					47	27	100	630	0 063
					46	25	100	570	0 057

* Catalog of Byron Jackson Iron Works, Inc., San Francisco, Cal.
† See Fig. 308, page 545. Remainder of table on next page.

Table 170. Discharges from Lawn Sprinklers* and Garden Hose Nozzles.— *(Continued)*

No. (see Fig. 308)	Name and type	Diameters, in.		No. of outlets	Pressure, lbs. per sq. in.		Length of ¾-in. hose, ft.	Discharge, gals. per hr.	Value of water per hr. at 10 cts. per 1000 gals.
		Inlet	Outlet		At sillcock	At nozzle			
10	Hotchkiss, whirl.	0.42	0.055	15	54	29	50	390	0.039
					30	25	50	270	0.027
					56	48	100	360	0.036
					36	25	100	240	0.024
11	Hotchkiss, fixed.	0.42	0.032	50	54	45	50	480	0.048
					36	25	50	390	0 039
					54	41	100	450	0.045
					33	25	100	390	0.039
12	No name, fixed	0.41	0.035	50	52	39	100	510	0.051
					36	25	100	420	0.042
					52	41	50	510	0.051
					35	25	50	420	0.042
13	Little Wonder, fixed.	0 40	0.32	1	54	49	50	360	0.036
					30	25	50	240	0 024
					53	44	100	330	0 033
					31	25	100	240	0 024
14	Niagara, fixed.	0.60	0.035	48	52	41	100	450	0.045
					36	25	100	390	0 039
					53	44	50	450	0.045
					32	25	50	390	0.039
15	Hero, fixed...	0 62	0 038	56	55	46	50	450	0 045
					34	25	50	330	0 033
					56	45	100	420	0 042
					33	25	100	300	0.030
16	Boston, jet......	0 563	0 22	1	60	55	50	210	0.021
					30	25	50	150	0 015
					60	55	100	210	0 021
					30	25	100	150	0.015
17	Boston, spray..	0 563	0.22	1	55	45	50	420	0 042
					30	25	50	300	0 030
					55	45	100	420	0.042
					32	25	100	330	0 033
18	Fairy, jet	0 600	0 18	1	57	48	50	450	0 045
					31	25	50	330	0.033
					57	45	100	450	0.045
					33	25	100	330	0 033
	Fairy, spray ..				65	60	50	150	0.015
					30	25	50	120	0 012
					61	59	100	180	0 018
					28	25	100	120	0 012
19	No name, jet ...	0 630	0 20	1	56	47	50	450	0.045
					31	25	50	330	0.033
					56	45	100	450	0.045
					33	25	100	330	0.033

* Tests by W. F. Sullivan, Pennichuck W. W., Nashua, N. H., Sept. 1, 1914. See Jl. N. E. W. W. Assn., Dec., 1914.

Table 171. Water Consumption in Foreign Cities*

*Information mainly from correspondence, July, 1913

City	Population supplied	Gals. per day per capita	Metered, % of consumption	Year	Consumption, mgd.	Remarks
Canada						
Montreal	600,000	153(a)	10.0	1912	80.4	(a) Municipal; 30% supply from private sources, 104 gal. per capita per day.
Toronto	425,000	118	22.5	1911	50.3	
Vancouver	125,000	173	———	1912	21 6	All but domestic supplies metered.
Winnipeg	225,000	56	———	1912	———	Large % metered.
South America						
Buenos Aires	1,252,000	35	100.0	1913	43 3	
Lima	175,000	57	None	1913	10 0	Estimated.
Montevideo	363,000	11	100.0	1913	4 1	Private company.
Rio Janiero	1,000,000	60	10.0	1912	59.8	
Sao Paulo	332,000	63	———	1912	21.2	50% services metered.
Great Britain						
Belfast	400,000	48	20.0	1912	19.2	
Birmingham	852,000	32	33.5	1913	26.9	
Dublin	307,000	46	22.0	1909	14 1	
Edinburgh	450,000	56	25.0	1912	25 2	
Glasgow	1,135,000	76	27.6	1913	86.5	Water area extends beyond city.
Liverpool	914,000	42	30.0	1912	38.8	Exclusive of 24 mgd. from private sources.
London	6,677,000	43	———	1913	28.4	23.3% of revenue comes from metered services.
Manchester	1,400,000	36	40.0	1913	50 4	Water area extends beyond city.
Northern Europe	(North of	50° Lat.)				
Amsterdam	566,000	26	100 0	1911	15 0	
Berlin	2,063,000	35	———	1911	70.0	25% supplied privately; 92% of public supply metered.
Brussels	312,000	25	100.0	1911	7.8	
Copenhagen	476,000	33	45.3	1913	15 6	
Dresden	555,000	30	72 4	1911	16 6	
Hague	292,000	20	50 0	1912	5.9	Some private supplies.
Hamburg	977,000	37	97.3	1912	36.0	
Leipsig	599,000	19	87 0	1912	11 4	
Moscow	————	———	100 0	1911	18 5	
Petrograd	2,018,000	38	86.5	1913	76 0	
Rotterdam	441,000	29	27.5	1912	12.6	Many private industrial supplies.
Stockholm	376,000	27	52.0	1912	10.0	
Warsaw	750,000	25	87.0	1912	18.6	
Southern Europe						
Athens	188,000	34	None	1913	6 3	Supply inadequate.
Buda Pest	910,000	58	39.3	1912	53 2	
Geneva	131,000	217	83.0	1912	28 4	
Madrid	570,000	84	25.0	1913	47 8	Of which 3% is private.

* Continued on next page.

Table 171. Water Consumption in Foreign Cities.—(*Continued*)

City	Population supplied	Gals. per day per capita	Metered, % of consumption	Year	Consumption, mgd.	Remarks
Southern Europe						
Marseilles.......	550,000	44	———	1912	24.5	70 large meters. Small private supplies.
Munich.........	615,000	75	———	1912	46.0	20%, private. 92% of public supply is metered.
Odessa..........	580,000	19	80.0	1912	11.0	
Paris...........	3,430,000	38	100.0	1911	111.0	Eng. News, Aug. 24, 1911.
Rome..........	542,000	120	100.0	1911	65.0	Private works.
Venice.........	132,000	40	65.0	1912	5.3	Private company.
Vienna........	2,065,000	25	50.0	1911	51.3	
Africa and Australia						
Alexandria......	420,000	28	50.0	1913	11.9	Private works. Natives take water from hydrants.
Cairo..........	705,000	25	———	1912	16.9	Population estimated.
Melbourne......	604,000	73	———	1912	44.4	30.7% of services metered.
Sydney.........	731,000	50	———	1912	36.4	20.7% of services metered.
Asia						
Bombay.......	979,000	37	———	1913	36.2	Some private supplies. 18.2% services metered.
Calcutta.......	1,109,000	62	2 0	1913	68.1	
Canton........	1,000,000	———	Small	1913	5.6	Used by only portion of population.
Osaka.........	1,148,000	17	———	1912	19.7	
Shanghai.......	488,000	———	9.3	1912	11.2	Population, 1910 French settlement supplied by private works, 14.13% of services metered.
Tokio	1,447,000	32	27 1	1912	46 0	

* Information regarding cities in U. S. is not given because readily available in other publications and the latest statistics are usually desired. See p. 545.

Table 172. Flow of Sink Faucets†

Name and description	Average pressure, lbs. per sq in: Static	Average pressure, lbs. per sq in: Stream flowing	Turns: 1/16	1/8	1/4	1/2	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/8	2 1/4	2 1/2
			Gallons per minute												
Mueller Standard‡ ¾ in. faucet, ⅜ in inlet and outlet. Total no. of turns, 2⅛.	63	59	2.1	5 4	7 1	11.5	19 0	22.3	23 0	23 2	23.5	23.8	24.1	....	....
	61	56	2.0	5 1	7 0	11 2	18 6	21 9	22.5	23 1	23.3	23.3	23.8	...	
Mueller extra, ½ in. faucet, ½ in. inlet and outlet. Total no. of turns, 2½	57	53	0.7	3 9	9 7	12 6	15.3	17 7	18 1	18 2	18.2	18 3	18 4	18.6	19.0
	58	54	0 8	4.0	10.3	12.8	15 5	18 6	18.2	18.2	18.2	18 6	18 6	18 7	19 0

† Tests by W. F. Sullivan, Pennichuck W. W., Nashua, N. H., Sept. 1, 1914. See Jl. N. E. W. W. Assn , Dec., 1914.
‡ Mueller Co.

CHAPTER XXV

HYDRAULIC COMPUTATIONS

Precision. For many hydraulic computations in practical waterworks problems, four-place logarithms, slide rules* and short-cut approximations are quite sufficient for the precision of the data and the necessities of the results. Use of many significant figures is a time-wasting absurdity, *e.g.*, giving total consumption of water, capacity of a reservoir, total pumpage, flow of a stream and similar quantities to the gallon when millions of gallons or tens of thousands would be appropriate, since inherent uncertainties commonly range from 5 to 25 per cent. Likewise most estimates of total cost should end with two to four ciphers before the decimal point.

In following formulas use feet and seconds.

Properties of Circular Pipes

(For contents of cylinders see p. 630.)

Hydraulic radius, **R**:

$$R = A \div P, \text{ where } P = \text{ wetted perimeter; } A = \text{area.}$$

For circular cross-section, full or half full,

$$R = \tfrac{1}{4}D \ (D = \text{diam.})$$

Table 176, p. 554, gives hydraulic radii of pipes, ½ to 144 in. diam.

Table 173. Velocity through Various Sized Pipes, Feet per Second

Diam., in	Gals. per min †													
	100	150	200	250	300	350	400	450	500	600	700	800	900	1000
3	4 54	6.81	9.08	11 35	13.62	...	....	.....	.....	....	.....	.....	....	....
4	2.55	3.83	5.11	6.38	7.66	8.94	10.21	11.49	12.77	....	....	.....	.....	.....
5	1 63	2.45	3.27	4.09	4.90	5 72	6.54	7.35	8.17	9.80	11.44	13.07	.....	.....
6	1.14	1.70	2.27	2.84	3.40	3.97	4.54	5.11	5.67	6.81	7.94	9.08	10.21	11.35
7	0.83	1.25	1.67	2.08	2.50	2.92	3.34	3.75	4.17	5.00	5.84	6.67	7.50	8.34
8	0.64	0 96	1.28	1.60	1.91	2.23	2.55	2.87	3.19	3.83	4.47	5.11	5.74	6 38
9	0.50	0 76	1.01	1.26	1.51	1.77	2.02	2.27	2.52	3.03	3.53	4.04	4.54	5 04
10	...	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.45	2.86	3.27	3.68	4 09
11		..	0.68	0 84	1.01	1.18	1.35	1.52	1.69	2.03	2.36	2.70	3 04	3.38
12		..	0.57	0 71	0.85	0.99	1.14	1.28	1.42	1.70	1.99	2.27	2.55	2 84
13	.	..	....	0 60	0.73	0 85	0.97	1.09	1.21	1.45	1.69	1.93	2.18	2 42
14			.	0 52	0.63	0.73	0.83	0.94	1.04	1.25	1.46	1.67	1.88	2.09
15		.	.	....	0.55	0.64	0.73	0.82	0.91	1.09	1.27	1.45	1.63	1 82
16				. ..	.....	0.56	0.64	0.72	0.80	0.96	1.12	1.28	1.44	1.60
18				...	....	....	0.50	0.57	0.63	0.76	0 88	1.01	1.14	1.26
20					....	....	.....	.....	0.51	0.62	0.72	0.82	0.92	1 02
24	.				.	..	.....	.....	.....	....	0.50	0.57	0.64	0 71
30	.					. ..	....	.....	.....	....	....	...	...	0 45

* Hydraulic slide rule, devised by Allen Hazen and Gardner S. Williams and on sale by dealers, is convenient for pipe computations and many other problems. † See page 552.

NOTE.—For more complete statements of theoretical hydraulics see standard books by Mansfield Merriman (John Wiley & Sons), Irving P. Church (John Wiley & Sons), Turneaure & Russell (John Wiley & Sons) and a recent treatise (1916), "Hydraulic Flow Reviewed," by Alfred A. Barnes (Spon & Chamberlain). The last contains some formulas with modified coefficients and exponents giving results in remarkably close agreement with experimental measurements for many pipes and channels.

Table 173. Velocity through Various Sized Pipes, Feet per Second.*—(*Continued*)

Diam., in.	Gals. per min.†												
	1250	1500	1750	2000	2500	3000	4000	5000	7500	10000	15000	20000	25000
6	14.18	.....	.....	....	..	.							.
7	10.42	12.51	14.59	16.67	....	...	.			.			.
8	7.98	9.57	11.17	12.77	....	....	....		..		.		
9	6.30	7.57	8.83	10.09	12.61	15.13	....	...					..
10	5.11	6.13	7.15	8.17	10.21	12.26	16.34	.....	.	.			
11	4.22	5 06	5.91	6.75	8.44	10.13	13.50	16.88		.	.	.	.
12	3.55	4.26	4.96	5.67	7.09	8.51	11.35	14.18	...	..	..	.	
13	3.02	3.63	4.23	4.83	6.04	7.25	9.67	12.09	18.13	..			
14	2 61	3.13	3.65	4.17	5.21	6.25	8.34	10.42	15 63	....	..	..	
15	2 27	2.72	3.18	3.63	4.54	5.45	7.26	9.08	13.62	18.16	..	.	
16	2 00	2.39	2.79	3.18	3.99	4.79	6.38	7.98	11.97	15.96	....	...	..
18	1.58	1.89	2.21	2.52	3.15	3.78	5.04	6.30	9.46	12.61	18.91	....	.....
20	1.28	1.53	1.79	2.04	2.55	3.06	4.08	5.11	7.66	10.21	15.32	..	....
24	0.89	1.06	1.24	1.42	1.77	2.13	2.84	3.55	8.32	7.09	10.64	14.18	....
30	0.57	0.68	0.79	0.91	1.14	1.36	1.82	2.27	3.40	4.54	6.81	9.08	11.35
36	....	.....	0.47	0.63	0.79	0.95	1.26	1.58	2.36	3.15	4.73	6.30	7 88
42	....	.....	.....	0.46	0 58	0 70	0.93	1.16	1.74	2.32	3.47	4.63	5 79
48	...	....	.....	.	0 44	0 53	0 71	0 89	1 33	1 77	2 66	3 54	4 43

(Theoretical; no allowance for tuberculation, inaccuracy in size, etc) * Byron Jackson Iron Works. † To use table for mgd., multiply quantity in mgd. by 700, and use this product as g p.m.

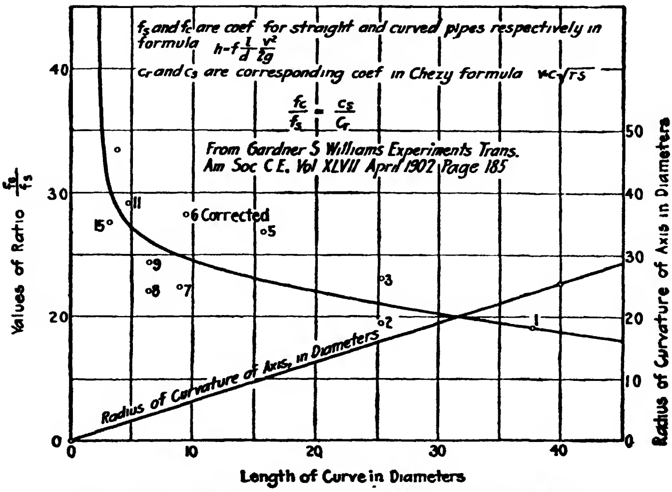


FIG. 309.—Loss of head in pipes at quarter bends.

Application of Fig. 309, which is based on a length of curve and tangent of 80 diams. What is loss of head in a 4″ quarter-bend; radius of curvature = 16″ = 4 diameters? On right-hand scale, Fig. 309, start at 4, move to left to straight line (Length of curve = 6.5 ± diams.), move up to curved line and read 2.6 on left-hand scale. $\frac{f_c}{f_s} = 2.6 = \frac{C_s}{C_r} \cdot C_r = C_s/2.6.$ Take C_s as 120 for clean *c. i.* pipe; so $C_r = 46$. Now $f_c = \frac{8g‡}{C^2} = \frac{8 \times 32.2}{46^2} = 0.122.$ $l_c = \pi d/4$. Loss of head on curve, $h_c = f_c \frac{l_c}{d}\frac{V^2}{2g} = \frac{0.122\,\pi d\, V^2}{4d \times 2g} = 0.030\,\pi\frac{V^2}{2g} = 0.0942\,\frac{V^2}{2g}.$ This loss is commonly taken at 0.15 $\frac{V^2}{2g}$.

‡ In formula $h = f\frac{l}{d}\frac{V^2}{2g}$; see Merriman's Treatise on Hydraulics, 1911, p. 216. In formula given by Church (Mechanics of Eng'g.), $h = 4f\frac{l}{d}\frac{V^2}{2g}$, $f = \frac{2g}{C^2}$. These expressions are the cause of much confusion.

Table 174. Head Required to Overcome Resistance in Circular bends of 90°, Exclusive of Friction*(Approximate)

For bend of less than 90°, divide resistance given by 90 and multiply by number of degrees of bend

Velocity feet per second	Ratio, or radius of bend at center of pipe / diameter of pipe								
	1	1.25	1.5	1.75	2	2.5	3	3.5	4
	Loss of head in feet								
1	0.004	0.003	0.003	0.002	0.002	0.002	0.002	0.002	0.002
2	0.018	0.013	0.010	0.009	0.009	0.008	0.008	0.008	0.008
3	0.041	0.029	0.024	0.021	0.020	0.019	0.019	0.018	0.018
4	0.073	0.051	0.042	0.038	0.036	0.034	0.033	0.033	0.033
5	0.114	0.080	0.066	0.060	0.056	0.053	0.052	0.052	0.051
6	0.164	0.115	0.095	0.086	0.081	0.077	0.075	0.074	0.074
7	0.224	0.156	0.130	0.117	0.111	0.105	0.102	0.101	0.101
8	0.292	0.204	0.169	0.153	0.144	0.137	0.134	0.132	0.131
9	0.370	0.259	0.214	0.194	0.183	0.173	0.169	0.167	0.166
10	0.456	0.319	0.265	0.239	0.226	0.214	0.209	0.206	0.205
11	0.552	0.386	0.320	0.289	0.273	0.258	0.253	0.250	0.248
12	0.657	0.460	0.381	0.344	0.325	0.308	0.301	0.297	0.296
13	0.771	0.540	0.447	0.404	0.381	0.361	0.353	0.349	0.347
14	0.895	0.626	0.519	0.469	0.442	0.419	0.409	0.405	0.402
15	1.027	0.719	0.596	0.538	0.508	0.481	0.470	0.465	0.462
16	1.169	0.818	0.678	0.612	0.578	0.547	0.535	0.529	0.526
17	1.319	0.923	0.765	0.691	0.652	0.617	0.603	0.597	0.593
18	1.480	1.035	0.858	0.775	0.731	0.692	0.677	0.670	0.665
19	1.648	1.153	0.956	0.863	0.815	0.771	0.754	0.746	0.741
20	1.826	1.278	1.059	0.957	0.903	0.855	0.835	0.826	0.821
21	2.013	1.408	1.167	1.055	0.996	0.942	0.921	0.911	0.906
22	2.209	1.546	1.281	1.157	1.093	1.034	1.011	1.000	0.994
23	2.415	1.690	1.400	1.265	1.194	1.130	1.105	1.093	1.086
24	2.629	1.840	1.525	1.377	1.300	1.231	1.203	1.190	1.183
25	2.853	1.996	1.654	1.495	1.411	1.335	1.305	1.291	1.284

Table 175. Hydraulic Elements of Circular Pipes, Part Full

Ratio of depth to diam.	Area of segment	Wetted perimeter	Hydraulic radius	Angle at center subtended by water surface
1.0	0.7854	3.1416	0.25	360°
0.95	0.7707	2.7085	0.285	309°40′
0.9	0.7445	2.4981	0.298	286°16′
0.8	0.6736	2.2140	0.3042	253°44′
0.7	0.5872	1.9824	0.296	227°10′
0.6	0.4920	1.7721	0.278	203°4′
0.5	0.3927	1.5708	0.25	180°
0.4	0.2934	1.3695	0.214	156°56′
0.3	0.1982	1.1592	0.171	132°50′
0.2	0.1118	0.9276	0.121	106°16′
0.1	0.0409	0.6434	0.0635	73°44′

Area of segment for any circle is found by multiplying diam. squared by coefficient found in "Area of segment" column, opposite ratio of depth to diam. For wetted perimeter and hydraulic radius, multiply coefficient, in respective columns, by diam.

* Byron Jackson Iron Works, San Francisco.

Table 176. Mathematical Properties of Cylindrical Pipes (Full)

Diam.		Powers of diam., D, ft					Circumference		Hydraulic radius = R, ft						Area		Contents per lin. ft *		
in.	ft., D	$1/D$	D^2	D^5	$D^{5/2}$	$\sqrt{D}$	in.	ft.	R	R^2	$R^{2/3}$	$1/R$	$\sqrt{R}$	$(1/R)^{4/3}$	sq. in.	sq. ft.	cu. ft.	gals.	lbs., water
½	0.0416	23.99	0.0017	0.0000001	0.00035	0.204	1.571	0.131	0.0104	0.0001	0.047	96.00	0.102	439.6	0.196	0.001	0.0014	0.0102	0.087
⅝	0.0521	19.19	0.0027	0.0000004	0.00062	0.228	1.964	0.164	0.0130	0.0002	0.055	76.80	0.114	326.4	0.307	0.002	0.0021	0.0159	0.181
¾	0.0625	16.00	0.0039	0.000001	0.0010	0.250	2.356	0.196	0.0156	0.0002	0.062	64.00	0.125	256.0	0.442	0.003	0.0031	0.0230	0.194
1	0.0833	12.00	0.0069	0.000004	0.0020	0.289	3.142	0.262	0.0208	0.0004	0.076	48.00	0.144	174.5	0.785	0.005	0.0055	0.0408	0.344
1¼	0.1042	9.60	0.0108	0.000012	0.0035	0.322	3.927	0.327	0.0260	0.0007	0.088	38.40	0.161	129.5	1.227	0.008	0.0085	0.0638	0.531
1½	0.1250	8.30	0.0156	0.00003	0.0055	0.354	4.712	0.393	0.0312	0.0010	0.099	32.00	0.177	101.6	1.767	0.012	0.0123	0.0918	0.769
1¾	0.1458	6.86	0.0213	0.00007	0.0081	0.382	5.500	0.458	0.0365	0.0013	0.110	27.43	0.191	82.7	2.405	0.017	0.0167	0.1249	1.043
2	0.1667	6.00	0.0279	0.00013	0.0113	0.408	6.283	0.524	0.0417	0.0017	0.120	24.00	0.204	69.2	3.142	0.022	0.0218	0.1632	1.36
2½	0.2083	4.8	0.0434	0.00039	0.01981	0.457	7.854	0.655	0.0521	0.0027	0.139	19.20	0.228	51.4	4.909	0.034	0.0341	0.2550	1.59
3	0.2500	4.00	0.0625	0.00098	0.03125	0.500	9.425	0.785	0.0625	0.0039	0.157	16.00	0.250	40.3	7.069	0.049	0.0491	0.3672	3.06
4	0.3333	3.00	0.1111	0.00412	0.06415	0.577	12.566	1.047	0.0833	0.0069	0.190	12.00	0.289	27.5	12.566	0.087	0.0873	0.6528	5.45
5	0.4167	2.40	0.1736	0.0126	0.1121	0.645	15.708	1.309	0.1042	0.0108	0.221	9.600	0.323	20.4	19.635	0.136	0.1364	1.020	8.53
6	0.5000	2.00	0.2500	0.0312	0.1768	0.707	18.850	1.571	0.125	0.0156	0.250	8.000	0.354	16.00	28.274	0.196	0.1963	1.469	12.27
8	0.6667	1.50	0.4444	0.132	0.3629	0.817	25.133	2.094	0.1667	0.0279	0.303	6.000	0.408	10.90	50.265	0.349	0.3491	2.611	21.8
10	0.8333	1.2	0.6944	0.402	0.6339	0.913	31.416	2.618	0.2083	0.0433	0.351	4.800	0.456	8.10	78.540	0.545	0.5454	4.080	34.0
12	1.000	1.0	1.0	1.0	1.00	1.0	37.699	3.142	0.25	0.0625	0.397	4.000	0.500	6.35	113.100	0.785	0.7854	5.875	49.1
14	1.166	0.857	1.3611	2.16	1.4702	1.080	43.982	3.665	0.2917	0.085	0.440	3.442	0.540	5.20	153.94	1.069	1.069	7.997	66.8
16	1.333	0.75	1.77	4.22	2.0528	1.155	50.266	4.189	0.3333	0.111	0.481	3.000	0.577	4.33	201.06	1.396	1.396	10.44	87.3
18	1.50	0.66	2.25	7.594	2.7557	1.224	56.549	4.712	0.3750	0.141	0.520	2.667	0.612	3.70	254.47	1.767	1.768	13.22	110.5
20	1.66	0.60	2.77	12.858	3.5861	1.291	62.832	5.236	0.4167	0.174	0.558	2.400	0.646	3.21	314.16	2.182	2.182	16.32	136.4
24	2.0	0.50	4.0	32.0	5.6569	1.414	75.398	6.283	0.5000	0.250	0.630	2.000	0.707	2.52	452.39	3.142	3.142	23.50	194
30	2.5	0.40	6.25	97.656	9.8821	1.581	94.248	7.854	0.6250	0.391	0.731	1.600	0.790	1.87	706.86	4.909	4.909	36.72	306
36	3.0	0.33	9.0	243.06	15.588	1.732	113.097	9.425	0.7500	0.563	0.826	1.333	0.866	1.47	1017.90	7.069	7.069	52.88	442
40	3.33	0.30	11.11	411.52	20.286	1.825	125.664	10.472	0.8333	0.694	0.885	1.200	0.913	1.28	1256.6	8.727	8.727	65.28	546
42	3.5	0.285	12.25	525.22	22.918	1.871	131.947	10.996	0.8750	0.766	0.915	1.143	0.935	1.19	1385.4	9.621	9.621	71.97	601
48	4.0	0.25	16.0	1024	32.0	2.0	150.796	12.566	1.000	1.000	1.000	1.000	1.000	1.00	1809.6	12.566	12.566	94.00	786
54	4.5	0.222	20.25	1845	42.957	2.121	169.646	14.137	1.125	1.263	1.081	0.888	1.060	0.854	2290.2	15.904	15.90	118.95	994
60	5.0	0.20	25.0	3125	55.902	2.236	188.496	15.708	1.250	1.563	1.161	0.800	1.118	0.74	2827.4	19.635	19.63	146.88	1227
66	5.5	0.182	30.25	5032	70.943	2.345	207.345	17.279	1.375	1.891	1.237	0.727	1.173	0.653	3421.2	23.758	23.76	177.72	1485
72	6.0	0.166	36.0	7776	88.182	2.449	226.195	18.850	1.5	2.250	1.310	0.667	1.225	0.583	4071.5	28.274	28.27	211.51	1767
78	6.5	0.154	42.25	11603	107.72	2.550	245.044	20.420	1.625	2.641	1.382	0.615	1.275	0.523	4776.4	33.183	33.18	248.23	2074
84	7.0	0.143	49.0	16807	129.64	2.646	263.894	21.991	1.75	3.063	1.453	0.571	1.323	0.474	5541.8	38.485	38.48	287.88	2405
90	7.5	0.133	56.25	23730	154.05	2.739	282.743	23.562	1.875	3.516	1.521	0.533	1.369	0.432	6361.7	44.179	44.18	330.48	2761
96	8.0	0.125	64.0	32768	181.02	2.828	301.593	25.133	2.000	4.000	1.587	0.5	1.414	0.397	7238.2	50.265	50.27	376.01	3141
102	8.5	0.118	72.25	44371	210.64	2.915	320.442	26.704	2.125	4.516	1.653	0.4706	1.458	0.366	8171.3	56.745	56.75	424.48	3546
108	9.0	0.111	81.0	59049	243.00	3.0	339.292	28.274	2.25	5.063	1.717	0.4444	1.500	0.339	9160.9	63.617	63.62	475.89	3976
114	9.5	0.105	90.25	77378	278.17	3.082	358.142	29.845	2.375	5.641	1.780	0.4210	1.541	0.316	10207.0	70.882	70.88	530.24	4430
120	10.0	0.10	100.0	100000	316.23	3.162	376.991	31.416	2.500	6.250	1.842	0.4000	1.581	0.295	11309.7	78.540	78.54	587.52	4910
126	10.5	0.095	110.25	127628	357.25	3.240	395.841	32.987	2.625	6.891	1.903	0.3810	1.620	0.276	12469.0	86.590	86.59	647.74	5410
132	11.0	0.091	121.0	161051	401.31	3.317	414.690	34.558	2.75	7.563	1.963	0.3636	1.658	0.259	13684.8	95.033	95.03	710.90	5940
138	11.5	0.087	132.25	201137	448.48	3.391	433.540	36.178	2.875	8.266	2.022	0.3478	1.696	0.244	14957.1	103.869	103.87	776.99	6490
144	12.0	0.083	144.0	248832	498.83	3.464	452.389	37.699	3.000	9.000	2.080	0.3333	1.732	0.231	16286.0	113.097	113.10	846.03	7070

* A pipe 1 yd. long holds approximately as many pounds of water as the square of its diam. in inches. (See also p. 630).

GENERAL FORMULAS FOR FLOW

Chezy formula, see p. 265.

"Long" and "Short" Pipes. Be careful not to take from tables or formulas for "long" pipes, discharges, etc., for pipes of insufficient length to be classed as "long." In such cases use formula for "short" pipes. $Q = CA\sqrt{2gh}$. Q = discharge, *cfs;* A = area, sq. ft.; g = acceleration of gravity; h = head on center of pipe, ft. Coefficient C varies with ratio of pipe diam., d, to length, l; if $\frac{l}{d} = 1$, $C = 0.62$; $\frac{l}{d} = 3$, $C = 0.815$; $\frac{l}{d} = 25$, $C = 0.71$; $\frac{l}{d} = 100$, $C = 0.55$. To be "long," a pipe should have length in excess of 500 diam.; if length is less than 50 diam., pipe is "very short." See also p. 556.

Table 177. Least Lengths of Pipes Considered "Long"

Diam., in.	1	2	3	4	6	8	10	12	16	20	24	30
Length, ft.	42	83	125	167	250	330	420	500	670	840	1000	1250

Diam.	36	42	48	60	72	84	96	108	120
Length.	1500	1750	2000	2500	3000	3500	4000	4500	5000

Boston Formulas for Service Pipes:

$$S = C_f\, V^2 \left(\frac{1}{R}\right)^{\frac{5}{4}} \quad V = C'_f R^{\frac{5}{8}} S^{\frac{1}{2}} \quad Q = AV$$

Kind of pipe	C_f	C'_f
Copper and lead,	0.00003	183
New, uncoated cast iron,	0.00005	141
Old cast iron, small tubercles,	0.00009	105
Old cast iron, large tubercles,	0.00015	82

Use Table 176 and tables of squares and cube roots of numbers.—(Boston W. W. Data Bk., 1895.)

Foss Formula:

$$S = C_f\, Q^{\frac{11}{6}}$$

S = Fric. head per ft.; C_f = coefficient varying with roughness and diam.; Q = discharge in cu. ft. per sec. With Q or diam. in ft. given, find $Q^{\frac{11}{6}}$ or C_f from Tables 180 and 178, or *vice versa.* Two of the quantities, S, $Q^{\frac{11}{6}}$ and C_f being known, third can be found by multiplication or division. Values of $C_f = \left(\frac{0.00065}{D^5}\right)$ in Table 178 are for coated new cast-iron and riveted steel or

Table 178. Foss Coefficients for Flow of Water in Pipes

Diam., D		C_f	Diam., D		C_f
in.	ft.		in.	ft.	
4	0.33	0.158	24	2.00	0.0000203
6	0.50	0.0208	30	2.50	0.0000067
8	0.67	0.00494	36	3.00	0.0000027
10	0.83	0.00162	40	3.33	0.0000016
12	1.00	0.00065	42	3.50	0.0000013
16	1.33	0.000154	48	4.00	0.0000006
20	1.67	0.0000506	....	......	....

Table 179. Relation of Kutter's n and Foss' C_f

Kutter's n	Multiply C_f by
0.009	0.65
0.010	0.80
0.011	1.00
0.012	1.25
0.013	1.50
0.015	1.90
0.017	2.50

sheet-iron pipes, flowing full (Kutter's n = 0.011). For circular conduits of other degrees of roughness multiply C_f by proper number in Table 179. Foss' formula for flow in channels is given on p. 265.

Table 180. Values of $Q^{\frac{11}{6}}$ (Foss formula)

Q	$Q^{\frac{11}{6}}$	Q	$Q^{\frac{11}{6}}$	Q	$Q^{\frac{11}{6}}$	Q	$Q^{\frac{11}{6}}$
0.01	0.0002	0.28	0 097	1.4	1.85	8 0	45.
0.02	0.0008	0.30	0.110	1.5	2.10	9 0	56.
0 03	0.0016	0.35	0.146	1.6	2.37	10 0	68.
0.04	0.0027	0.40	0.18	1.8	2.94	11.0	81.
0.05	0.0041	0.45	0.23	2.0	3 56	12.0	95.
0.06	0.0057	0.50	0 28	2 2	4.24	14 0	126.
0 07	0.0076	0.55	0.33	2.4	4.98	16 0	161.
0.08	0 0097	0 60	0 39	2.6	5.76	18.0	200.
0 09	0.0121	0.65	0 45	2.8	6.6	20 0	242.
0 10	0.0146	0 70	0.52	3.0	7.5	22.0	289.
0.12	0 0205	0 75	0 59	3 5	9 9	25 0	365.
0 14	0.0272	0.80	0.66	4 0	12 7	30 0	511.
0 16	0 0347	0 90	0 82	4 5	15.8	40 0	865.
0 18	0.0431	1.00	1.00	5 0	19 1	60 0	1,819.
0.20	0 052	1.1	1 19	5 5	22.8	80.0	3,083.
0 22	0.062	1 2	1.40	6 0	26.	100.0	4,641.
0 24	0 073	1 3	1.62	7 0	35	200 0	16,540
0 26	0 085	......	....	.. .	. .		

(W. E. Foss, J. A. E. S., Vol. 13, p. 295, Mar. 21, 1894)

Friction Head in Long Pipes. For a steady flow of water in stationary, rigid cylindrical pipes, the loss of head due to fluid friction is conveniently expressed: $H_f = \dfrac{4fLV^2}{D2g}$ where L is the length on slope, D, the internal diam. of the pipe, V the mean velocity (component parallel to axis of pipe) of the particles of water passing through any given cross-section (generally about 83 per cent. of the velocity of particles near the center of the section), and f = the coefficient of fluid friction.* Results from Fig. 322 differ but slightly from Metcalf's "Diagram D" (E. R., June 20, 1903); see also p. 317, Fig. 568, for general use with new cast-iron pipes, based on the Hazen-Williams formula: (mean velocity) $V = 71.66D^{\frac{63}{100}} s^{\frac{54}{100}}$, in which $s = \dfrac{H_f}{L}$. According to Metcalf, H_f, for old and tuberculated pipe, for a given mean velocity = $\frac{183}{100}$ of that given by the diagram for clean cast-iron pipes; for a given friction head, the velocity from the diagram for clean cast-iron pipe must be multiplied by $\frac{13}{18}$ to give velocity for tuberculated pipe. Similarly, Metcalf, for riveted pipe, finds that H_f from diagram for clean cast-iron pipe must be multiplied

* Values of f varying with D and V are given in "Mechanics of Engineering" (I. P Church) and elsewhere.

by $\frac{138}{100}$ for the same diam. and velocity; or, conversely, the mean velocity from the diagram for clean cast-iron pipe must be multiplied by $\frac{12}{43}$.—(I. P. Church, Hydraulic Motors, 1908.)

General Form of Equation for Pipe Friction. A. V. Saph and E. W. Schoder (T. A. S. C. E., Vol. 51, 1903, p. 257) investigated, by logarithmic plotting of the observed losses of head with the corresponding velocities, experiments in which the velocities ranged from a fraction of a foot per sec. up to 44 ft. Their own experiments, almost 800 and including 23 kinds of pipe and hose, covered velocities from 0.1 to 22 ft. per sec. In all, and especially in those of known accuracy in all measurements, the logarithmic plottings yielded straight lines; the slopes of these lines differ considerably, or, in other words, the losses of head vary as powers of the velocities, but whatever the slope of the line, each pipe appears to have a definite law, expressible in the form, $H_f = MV^n$. M and n are constants for a given pipe in a given condition for all velocities. For

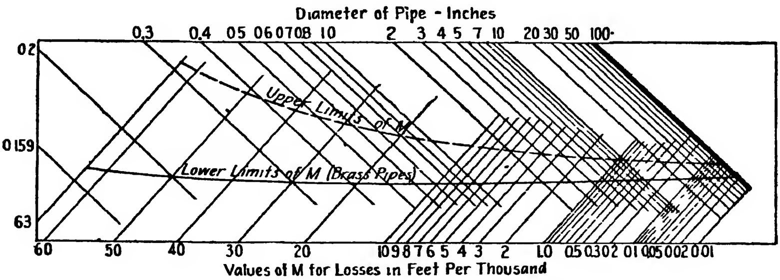
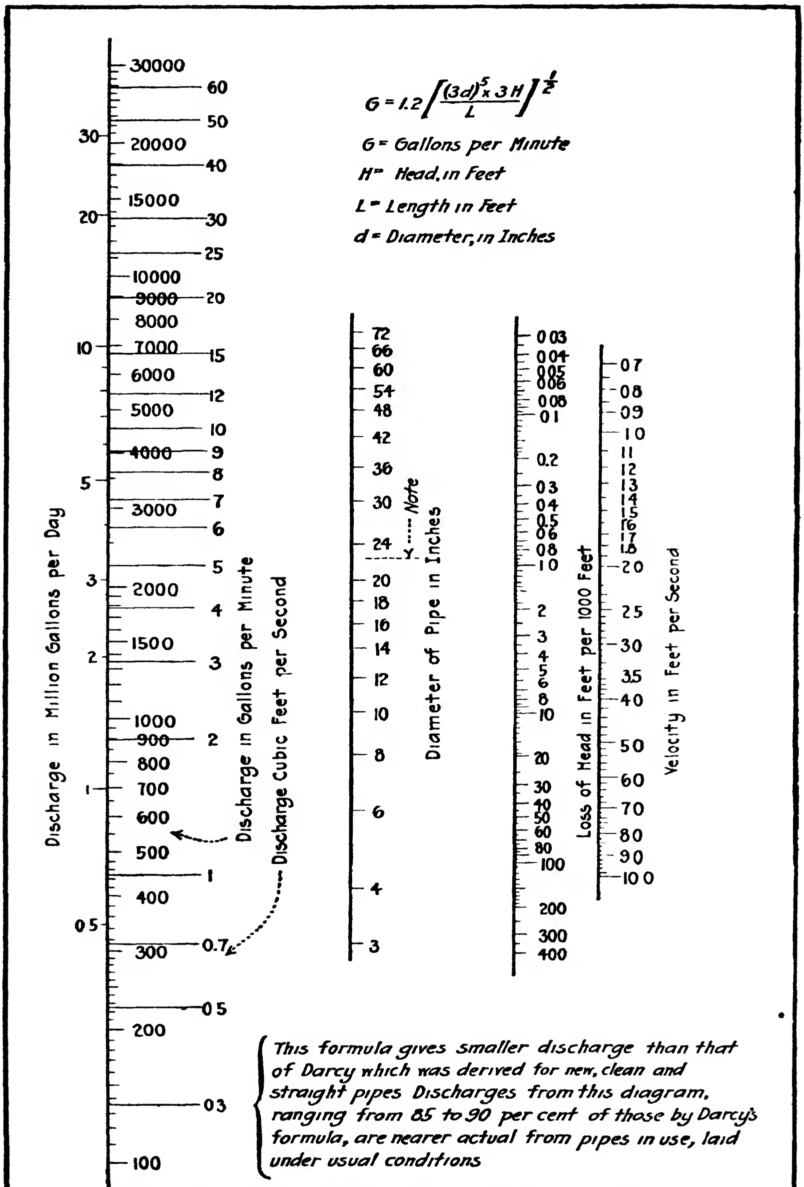


FIG. 310—Flow of water in pipes, 0 2″ to 100.

(Saph and Schoder's diagram* reduced by plotting to logarithmic scale distances scaled from their log diagram)

brass pipe, representing extreme smoothness and ideal conditions, $M = \frac{0.296}{D^{1.25}}$. M is in feet per thousand, and D in feet. (See Fig. 310.) Using the average value of n there results, $H_f = \frac{0.296}{D^{1.25}} V^{1.75}$. H_f, is in feet per thousand. With the maximum variations in the value of M of approximately 7 per cent. each way from this equation, the loss of head in extremely smooth pipes laid straight is, $H_f = \frac{0.296}{D^{1.25}} V^{1.75} \pm 7$ per cent.; whence $V = 104\, s^{0.57} D^{0.71} \pm 4$ per cent., or using hydraulic radius, R, $V = 279\, s^{0.57} R^{0.71} \pm 4$ per cent. s = hydraulic slope in feet, per foot of length. In these expressions, *temperature* effect has not been included; its effect is to vary the loss of head about 4 per cent. for a change of 10° F. Owing to the uncertainty regarding the effect on the rougher pipes used in practice, its insertion might be misleading. For Noble's 44-in. wood pipe, p. 571, $M = \frac{0.687}{D^{1.25}}$. The variation in n will be considered as from 1.82 to 1.99, and the general formula becomes $H_f = \frac{0.687}{D^{1.25}} V^{1.82 \text{ to } 1.99}$; $V = 54.7 \text{ to } 38.9\, s^{0.55 \text{ to } 0.50}\, D^{0.69 \text{ to } 0.63} = 142 \text{ to } 93\, s^{0.55 \text{ to } 0.50}\, R^{0.69 \text{ to } 0.63}$.

*T. A. S. C. E., Vol. 51, 1903, P. XI



Note — *This formula agrees closely with experiments on pipes in use, less than 24 inches diameter, but gives too small discharges for old pipes larger than 24 inches under ordinary conditions of foulness,—for correction see Fig. 312. Use of Diagram: Lay a straight edge to connect any two variables; the intercepts with other vertical lines give corresponding values of the other two variables.*

(*Fred F. Moore.*)

FIG. 311.—Flow in Cast-iron pipes. Box's formula.

(See also Turneaure and Russell, "Public Water Supplies.")

These formulas may be regarded as giving the two extremes, the first for very smooth pipes laid very straight, the second for a considerable degree of tuberculation (or inner surface comparable to it). The greater majority of cases which confront the hydraulic engineer are included in the values of n from 1.74 to 2.00, whence,

$$H_f = \frac{0.296 \text{ to } 0.469}{D^{1.25}} V^{1.74 \text{ to } 2.00}$$

$$V = 107 \text{ to } 46\ s^{0.57 \text{ to } 0.50} D^{0.72 \text{ to } 0.62}$$
$$= 289 \text{ to } 109\ s^{0.57 \text{ to } 0.50} R^{0.72 \text{ to } 0.62}$$

The last formulas represent the range of results which the best modern experimenters have obtained on actual pipe lines in service or ready for service, when the critical velocity is not exceeded. Under no conditions will n exceed 2.0.

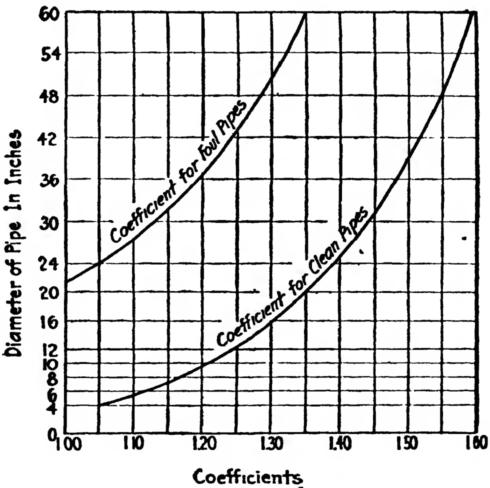


FIG. 312.—Coefficients for Fig. 311. Multiply discharges by these coefficients to get results for clean or foul pipes. (*F. F. Moore.*)

In 1873 Dr. Lampe presented his experiments on the Danzig pipe line, with the formula: $V = 203.3\ R^{0.694}\ S^{0.555}$. Starting with the formula $H = mV^n$, m and n being constants for any given pipe line, G. S. Williams attempted to discover the effect upon the exponent n due to changes of diam., alinement and character of the interior surface. From an investigation of more than 80 series of experiments by 13 observers, the following conclusions were drawn: (1) n increases as the diam. increases, from about 1 in capillary tubes to about 2 in large pipes, being from 1.8 to 2 in those ordinarily used; (2) n increases as the roughness increases. (3) n decreases as the curvature increases; (4) n is different for different materials, being lowest for tin and brass. A. V. Saph and E. W. Schoder in observations on seamless brass tubing, 0.1 to 2.1 in. diam., show n to be practically constant and approximately 1.75 for such pipe. Experiments by T. A. Noble, also give about 1.75, which, taken with Saph and Schoder, seems to establish this value

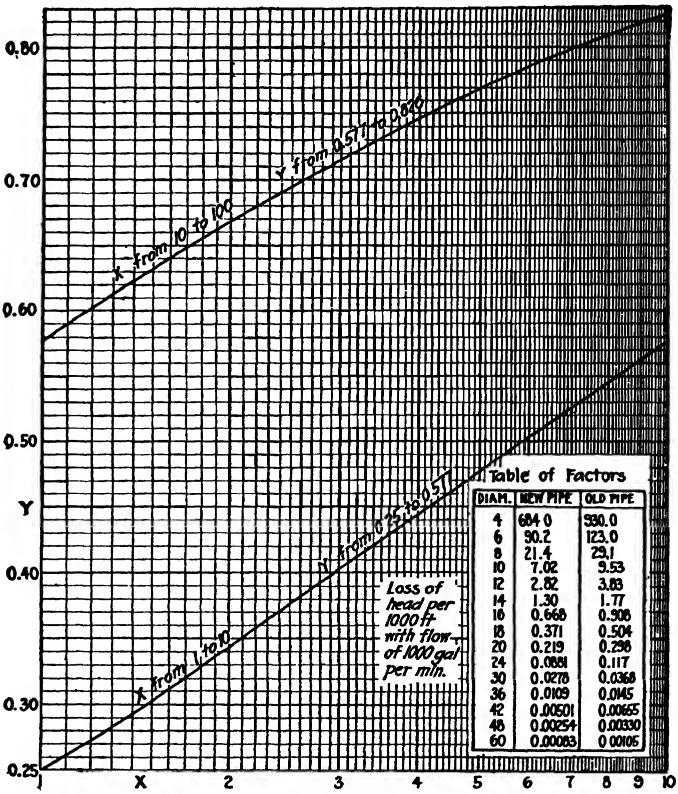
for smooth pipe, and proves in error conclusion (1); as long as the pipe remains smooth, n does not change.—(T. A. S. C. E., Vol. 49, 1902, p. 156.)

Effect of Condition of Pipe on Friction Head. Text-books make the general statement that the friction head in pipes varies nearly as the square of the velocity. Researches indicate that this exponent varies between 1.75 and 2.00.

Based on formula $y = \dfrac{x}{(1+\sqrt{x})^2}$

$$x = \frac{\text{Factor of pipes of greatest resistance} \times \text{length}}{\text{Factor of pipes of least resistance} \times \text{length}}$$

$y =$ Coefficient by which f_s must be multiplied to obtain factor of equivalent pipe.



DIAM.	NEW PIPE	OLD PIPE
4	684 0	930.0
6	90.2	123.0
8	21.4	29.1
10	7.02	9.53
12	2.82	3.83
14	1.30	1.77
16	0.668	0.908
18	0.371	0.504
20	0.219	0.298
24	0.0881	0.117
30	0.0278	0.0368
36	0.0109	0.0145
42	0.00501	0.00665
48	0.00254	0.00330
60	0.00083	0 00105

FIG. 313.—Equivalents of parallel pipes. (F. F. Moore.)

By means of the diagram for determining "equivalents of parallel pipes," the discharging capacity at any point of a system of connecting pipes may be readily computed by first ascertaining the sizes and lengths of an hydraulically equivalent, single, compound pipe line from the source of supply to the point of discharge. This is accomplished by successively combining a derived equivalent pipe with another pipe of the system. The combinations for the successive computations of the equivalents must be so selected with reference to the physical conditions that total losses of head in the two pipes are the same. For example, consider two pipes leading from a reservoir to the same point in a distribution system; a 10-in. 6,000 ft. long and an 18-in. 10,000 ft. long, or the problem is the same if these pipes branch below the reservoir; required the diameter of the equivalent single pipe 10,000 ft. long. For most cases of pipes in use the factors for "old pipe" will give results nearer the truth. Then

$$x = \frac{f_l \times \text{length in thousands of feet}}{f_s \times \text{length in thousands of feet}} = \frac{9.53 \text{ (from table)} \times 6}{0.504 \text{ (from table)} \times 10} = 11.3$$

y (for $x = 11.3$) $= 0.594$ (from diagram); $0.594 \times 0.504 = 0.299$, which is the factor for a 20-in. pipe (nearly).

The general formula is $H_f = K\frac{V^n}{D^p}$

Condition of pipe	H_f	V
Very smooth pipes (seamless drawn brass tubes)	$0.30\frac{V^{1.75}}{D^{1.25}}$	$1.99\ H_f^{0.57}D^{0.71}$
Ordinary pipes	$0.38\frac{V^{1.86}}{D^{1.25}}$	$1.69\ H_f^{0.54}D^{0.67}$
Foul or tuberculated pipes	$0.69\frac{V^{2}}{D^{1.25}}$	$1.20\ H_f^{0.50}D^{0.62}$
Lap-riveted steel pipes, several years old	$0.50\frac{V^{1.95}}{D^{1.25}}$	$1.42\ H_f^{0.51}D^{0.64}$

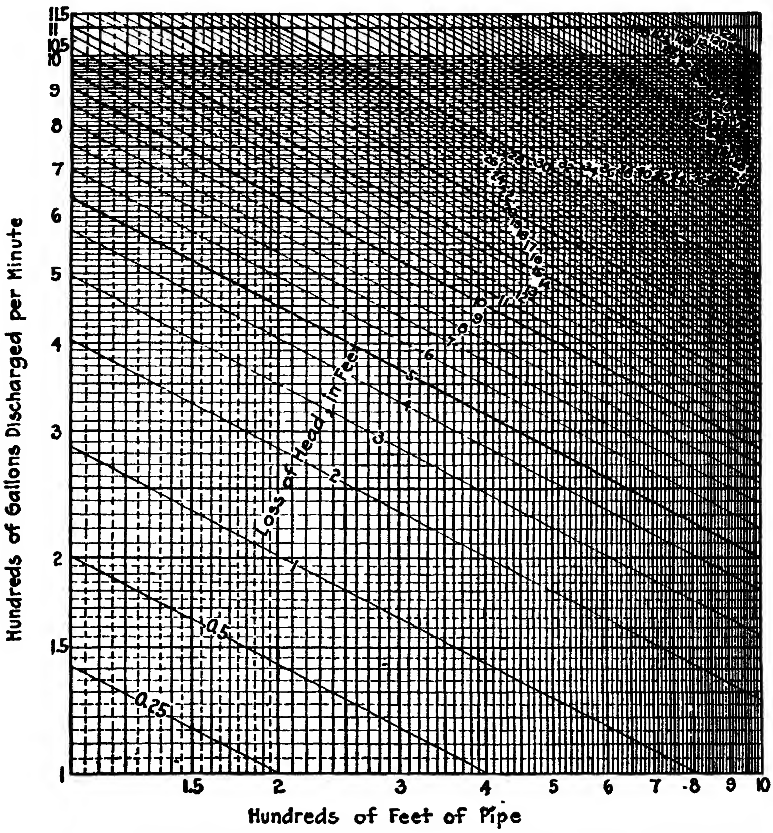


FIG. 314.—Flow from 6-in. c. i. pipe, Box's formula. (*Geo. W. Booth.*)

Probably no wise engineer would attempt to predict the discharge of a 5-year-old pipe line within 10 per cent. The above formulas will be on the side of the truth, and within 20 per cent. thereof.—(E. W. Schoder, Cornell Civil Engineer, May, 1910.)

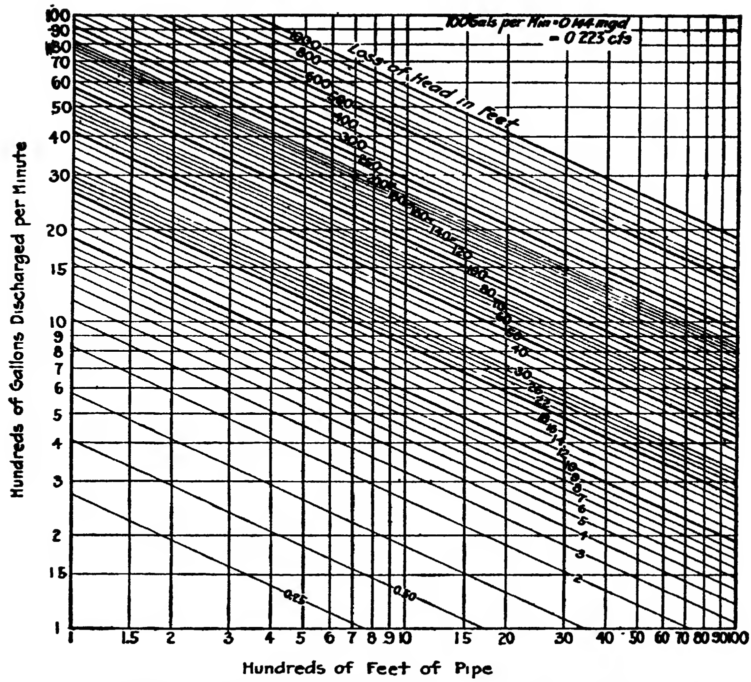


FIG. 315.—Flow from 8-in. c. i. pipe, Box's formula. (*Geo. W. Booth.*)

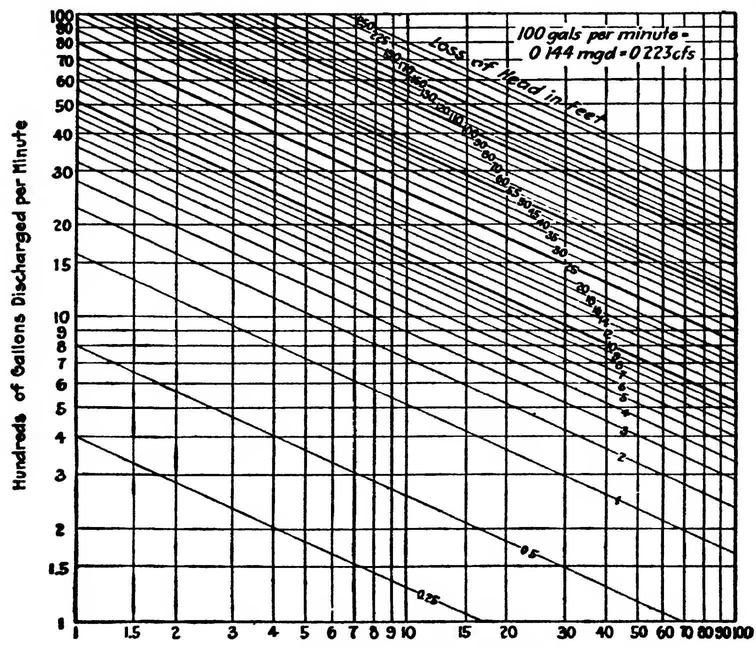


FIG. 316.—Flow from 12-in. c. i. pipe, Box's formula. (*G. W. Booth.*)

Table 181. Flow of Water in New and Old Cast-iron Pipes of Ordinary Sizes in Distribution Systems
Quantities, Velocities, and Friction Losses

Diam. in	Vel. = 3 ft. per sec.			
	Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft
4	0.169	0.262	**310.** / 55.	**59.** / 10.4
6	0.381	0.589	**157.** / 34.3	**29.8** / 6.5
8	0.677	1.047	**100.** / 24.8	**18.8** / 4.70
10	1.058	1.636	**71.** / 19.4	**13.4** / 3.68
12	1.523	2.356	**54.** / 15.9	**10.2** / 3.02
14	2.073	3.207	**43.4** / 13.83	**8.2** / 2.62
16	2.707	4.189	**35.9** / 11.7	**6.8** / 2.22
18	3.426	5.30	**30.5** / 10.3	**5.8** / 1.96
20	4.230	6.54	**26.5** / 9.24	**5.0** / 1.75
24	6.09	9.42	**20.8** / 7.7	**3.95** / 1.45
30	9.52	14.73	**15.7** / 6.1	**2.97** / 1.15
36	13.71	21.21	**12.5** / 5.0	**2.37** / 0.95
40	16.92	26.181	**11.0** / 4.5	**2.09** / 0.85
42	18.66	28.86	**10.4** / 4.28	**1.97** / 0.81
48	24.37	37.70	**8.9** / 3.75	**1.68** / 0.71
54	30.84	47.71	**7.6** / 3.32	**1.46** / 0.63
60	38.07	58.9	**6.9** / 3.01	**1.30** / 0.57
66	46.07	71.3	**6.2** / 2.69	**1.17** / 0.51
72	54.8	84.8	**5.6** / 2.48	**1.06** / 0.47

Diam. in	Vel. = 4 ft per sec			
	Mil gals. per day	Cu. ft per sec.	Friction head per mile	Friction head per 1000 ft.
4	0.226	0.34	**553.** / 99.	**105.** / 187.
6	0.508	0.78	**281.** / 61.	**53.** / 11.6
8	0.902	1.39	**178.** / 44.3	**33.7** / 8.4
10	1.410	2.18	**126.** / 34.7	**23.9** / 6.6
12	2.031	3.14	**96.** / 28.5	**18.3** / 5.4
14	2.763	4.27	**78.** / 24.1	**14.7** / 4.57
16	3.601	5.58	**64.** / 20.93	**12.2** / 3.96
18	4.569	7.07	**55.** / 18.5	**10.3** / 3.50
20	5.64	8.73	**47.4** / 16.5	**9.0** / 3.13
24	8.12	12.57	**37.8** / 13.7	**7.0** / 2.59
30	12.69	19.63	**28.0** / 10.9	**5.3** / 2.06
36	18.28	28.27	**22.4** / 9.0	**4.24** / 1.70
40	22.56	34.91	**19.7** / 8.1	**3.73** / 1.53
42	24.87	38.48	**18.6** / 7.7	**3.53** / 1.45
48	32.49	50.3	**15.9** / 6.7	**3.01** / 1.27
54	41.12	63.6	**13.8** / 6.00	**2.62** / 1.13
60	50.8	78.5	**12.2** / 5.3	**2.32** / 1.01
66	61.4	95.0	**11.0** / 4.86	**2.08** / 0.92
72	73.1	113.1	**10.0** / 4.44	**1.89** / 0.84

Diam. in	Vel. = 5 ft. per sec.			
	Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft
4	0.283	0.43	**863.** / 154.	**163** / 29.1
6	0.635	0.98	**438.** / 96.	**83.** / 18.1
8	1.129	1.74	**277.** / 69.	**53.** / 13.1
10	1.76	2.72	**197.** / 54.	**37.3** / 10.3
12	2.53	3.92	**151.** / 44.4	**28.5** / 8.4
14	3.45	5.34	**121.** / 37.6	**22.9** / 7.1
16	4.51	6.98	**100.** / 32.6	**18.9** / 6.2
18	5.71	8.83	**85.** / 28.8	**16.1** / 5.5
20	7.05	10.91	**74.** / 25.8	**14.0** / 4.89
24	10.15	15.71	**58.** / 21.3	**11.0** / 4.04
30	15.86	24.54	**43.7** / 16.9	**8.3** / 3.21
36	22.84	35.34	**34.9** / 14.0	**6.6** / 2.66
40	28.05	43.63	**30.3** / 12.6	**5.8** / 2.39
42	31.09	48.11	**29.0** / 12.0	**5.5** / 2.27
48	40.61	62.8	**24.8** / 10.4	**4.69** / 1.98
54	51.4	79.5	**21.5** / 9.3	**4.08** / 1.76
60	63.5	98.2	**19.1** / 8.3	**3.62** / 1.58
66	76.7	118.8	**17.2** / 7.5	**3.25** / 1.43
72	91.4	141.4	**15.5** / 6.9	**2.94** / 1.31

Diam. in	Vel = 6 ft. per sec.			
	Mil. gals. per day	Cu. ft per sec.	Friction head per mile	Friction head per 1000 ft.
4	0.34	0.52	**1244.** / 227.	**236.** / 42.
6	0.76	1.18	**631.** / 138.	**120.** / 26.1
8	1.35	2.10	**399.** / 100.	**76.** / 18.9
10	2.11	3.27	**284.** / 78.	**54.** / 14.8
12	3.05	4.71	**217.** / 64.	**41.1** / 12.1
14	4.15	6.41	**174.** / 54.	**33.0** / 10.3
16	5.42	8.39	**144.** / 47.0	**27.3** / 8.9
18	6.85	10.60	**123.** / 41.6	**23.2** / 7.9
20	8.46	13.09	**106.** / 37.2	**20.2** / 7.04
24	12.18	18.85	**84.0** / 30.7	**15.9** / 5.8
30	19.03	29.45	**63.** / 24.4	**11.9** / 4.62
36	27.41	42.41	**50.** / 20.2	**9.5** / 3.83
40	33.84	52.4	**44.3** / 18.1	**8.4** / 3.43
42	37.31	57.7	**41.5** / 17.3	**7.9** / 3.27
48	48.73	75.4	**35.7** / 15.0	**6.8** / 2.85
54	61.7	95.4	**31.0** / 13.4	**5.9** / 2.53
60	76.1	117.8	**27.6** / 12.0	**5.2** / 2.27
66	92.1	142.5	**24.7** / 10.9	**4.68** / 2.06
72	109.6	169.6	**22.4** / 10.0	**4.24** / 1.89

Diam. in	Vel. = 7 ft. per sec.			
	Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft
4	0.39	0.61	**1660.** / 302.	**314.** / 57.
6	0.88	1.38	**843.** / 188.	**160.** / 35.5
8	1.58	2.44	**533.** / 136.	**101.** / 25.7
10	2.46	3.82	**379.** / 106.	**72.** / 20.1
12	3.55	5.5	**289.** / 87.	**55.** / 16.5
14	4.83	7.48	**235.** / 74.	**44.1** / 14.0
16	6.32	9.77	**192.** / 64.	**36.4** / 12.1
18	7.99	12.37	**164.** / 57.	**31.0** / 10.7
20	9.87	15.27	**142.** / 51.6	**26.9** / 9.6
24	14.21	21.99	**112.** / 41.9	**21.2** / 7.9
30	22.21	34.36	**84.** / 33.2	**15.9** / 6.39
36	31.98	49.48	**67.** / 27.5	**12.7** / 5.2
40	39.48	61.1	**59.** / 24.5	**11.2** / 4.65
42	43.53	67.3	**56.** / 23.5	**10.6** / 4.45
48	56.8	88.0	**47.6** / 20.5	**9.0** / 3.88
54	71.9	111.3	**41.4** / 18.2	**7.9** / 3.44
60	88.8	137.4	**36.8** / 16.3	**7.0** / 3.09
66	107.5	166.3	**32.9** / 14.8	**6.2** / 2.81
72	127.9	197.9	**29.9** / 13.6	**5.7** / 2.57

The heavier type refers to old pipe. Computed from E. B. Weston's formula: New pipe, $H = \left(0.019892 + \frac{0.00166573}{D}\right)\frac{L}{D}\frac{V^2}{2g}$.

Old pipe, $H = \left(0.03978 + \frac{0.03332}{D}\right)\frac{L}{D}\frac{V^2}{2g}$. (T.A.S.C.E., Vol. 22, 1890, p. 60.) H = loss of head due to friction, ft. D = pipe diameter, ft. L' = pipe length, ft.

Table 182. Service Pipes—Approximate Quantity Delivered
(1877 Report of Rochester Waterworks)

Diam.	$H_f = 10L$			$H_f = 9L$			$H_f = 8L$		
in.	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0.0441	19.8	28500	0.0417	18 7	26900	0.0394	17.7	25500
⅝	0 0769	34.5	49700	0.0729	32.7	47100	0.0671	30.1	43300
¾	0.1212	54.4	78300	0.1152	51.7	74400	0.1085	48.7	70100
1	0.2491	112.	161000	0.2362	106.	153000	0.2228	100.	144000
1¼	0.435	195.	281100	0 413	185.	267000	0.389	175.	251000
1½	0 686	308.	443500	0 651	292.	421000	0.614	275.	397000
2	1 409	630.	910000	1 336	600.	864000	1.262	570.	816000
2½	2.460	1100.	1590000	2.335	1050.	1509000	2.201	990.	1422000
3	3.888	1750.	2513000	3.678	1650.	2377000	3.476	1560.	2246000
4	7.979	3580.	5157000	7.569	3400.	4892000	7.136	3200.	4612000
5	13 918	6200.	8996000	13.208	5900.	8536000	12.450	5600.	8047000
6	21 96	9900.	14191000	20.83	9300.	13463000	19.640	8800.	12692000

Diam	$H_f = 7L$			$H_f = 6L$			$H_f = 5L$		
in.	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0.0368	16.5	23800	0 0341	15 3	22000	0 0312	14 0	20200
⅝	0 0644	28 9	41600	0 0590	26 5	38200	0 0544	24 4	35100
¾	0.1016	45 6	65700	0.0940	42 2	60800	0 0858	38.5	55400
1	0.2083	93.5	135000	0.1929	86.6	125000	0.1760	79.	114000
1¼	0.364	163.	235000	0 337	151.	218000	0 308	138.	199000
1½	0 574	258.	371000	0 531	239.	343000	0 485	218.	313000
2	1.178	530.	762000	1 088	488.	703000	0 996	447.	644000
2½	2 059	920.	1331000	1.906	860.	1232000	1.740	780.	1124000
3	3.253	1460.	2102000	3.010	1350.	1945000	2.749	1230.	1777000
4	6.675	3000.	4314000	6 181	2770	3995000	5.641	2530.	3646000
5	11 646	5200.	7527000	10 781	4840.	6968000	9.841	4420.	6360000
6	18.37	8200.	11873000	17.006	7600.	10992000	15.525	6700.	10034000

	$H_f = 4L$			$H_f = 3L$			$H_f = 2L$		
½	0 0279	12.5	18000	0.0241	10 8	15600	0 0196	8 8	12700
⅝	0 0486	21.8	31400	0 0421	18 9	27200	0 0343	15 4	22200
¾	0 0766	34.4	49500	0 0664	29 8	42900	0.0541	24 3	35000
1	0 1575	70.7	102000	0 1364	61.2	88100	0.1114	50.0	72000
1¼	0 2749	123.	178000	0 2382	107.	154000	0.1945	87.3	126000
1½	0 434	195.	281000	0 376	169.	243000	0.307	138.	198000
2	0 891	400.	576000	0.772	346.	499000	0.630	283.	407000
2½	1.556	700.	1006000	1.348	600.	871000	1 098	493.	710000
3	2.458	1100.	1588000	2.129	960.	1376000	1 738	780.	1123000
4	5 046	2270.	3262000	2.366	1060.	1529000	3.569	1600.	2307000
5	8.803	3950.	5689000	7.589	3410.	4905000	6 218	2790.	4019000
6	13.887	6200.	8976000	12.011	5400.	7763000	9 819	4410.	6346000

	$H_f = 1\frac{3}{4}L$			$H_f = 1\frac{1}{2}L$			$H_f = 1\frac{1}{4}L$		
½	0 0185	8 3	12000	0 0172	7 7	11100	0.0156	7 0	10100
⅝	0.0321	14 4	20700	0.0299	13.4	19300	0 0272	12.2	17600
¾	0.0508	22 8	32800	0.0470	21.1	30400	0 0430	19 3	27800
1	0.1043	46.8	67400	0.0962	43.2	62200	0 0880	39.5	56900
1¼	0.1818	81.6	118000	0.1684	75.6	109000	0.1537	69.	99400
1½	0.2870	129.	185000	0.2658	119.	172000	0 2426	109.	157000
2	0.589	264.	381000	0.554	249.	358000	0 498	224.	322000
2½	1.029	462.	665000	0.953	428.	616000	0.870	390	562000
3	1.624	730.	1049000	1.504	670.	972000	1 372	620.	887000
4	3.333	1500.	2154000	3.086	1390.	1994000	2 816	1260.	1820000
5	5.822	2610.	3763000	5.392	2420.	3485000	4.922	2210.	3181000
6	9.184	4120.	5936000	8.504	3820.	5496000	7 762	3480.	5017000

Table 182. Service Pipes—Approximate Quantity Delivered.—(*Continued*)

Diam.	$H_f = L$			$H_f = \frac{3}{4}L$			$H_f = \frac{1}{2}L$		
in.	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0 0140	6.3	9070	0 0120	5.4	7780	0 0098	4 4	6340
⅝	0.0243	10.9	15700	0 0212	9 5	13700	0 0172	7 7	11100
¾	0.0383	17.2	24800	0 0332	14.9	21500	0 0272	12 2	17600
1	0.0786	35.3	50800	0.0682	30.6	44100	0 0557	25.0	36000
1¼	0 1375	61.7	88800	0.1192	53 5	77000	0 0974	43.7	63000
1½	0.2170	97.4	140000	0 1878	84.3	121000	0.1531	68.7	99000
2	0.445	200.	288000	0.386	173.	249000	0.315	141.	204000
2½	0.778	349.	503000	0 674	302.	435000	0.550	247.	356000
3	1.238	560.	800000	1 063	477.	687000	0 869	390.	562000
4	2.524	1130.	1632000	2.182	980.	1410000	1.784	800.	1153000
5	4.403	1980.	2845000	3 812	1710.	2464000	3.106	1390.	2007000
6	6 942	3120.	4487000	6.000	2690.	3878000	4 911	2200.	3174000

Diam.	$H_f = \frac{1}{3}L$			$H_f = \frac{1}{4}L$			$H_f = \frac{1}{5}L$		
in.	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0.0080	3.6	5180	0 0069	3 1	4460	0 0062	2.8	4030
⅝	0.0140	6 3	9070	0.0123	5.5	7920	0 0107	4 8	6910
¾	0 0221	9.9	14300	0.0192	8.6	12400	0.0172	7.7	11100
1	0.0455	20.4	29400	0.0394	17.7	25500	0 0352	15.8	22800
1¼	0.0793	35.6	51300	0 0688	30 9	44500	0 0615	27.6	39700
1½	0 1252	56.2	80900	0 1085	48 7	70100	0 0978	43 9	63200
2	0 2571	115.	166000	0 2228	100.	144000	0.1992	89 4	129000
2½	0 449	202.	290000	0.389	175.	251000	0.348	156.	225000
3	0 708	318	458000	0.615	276	397000	0 550	247.	355000
4	1 457	650.	941000	1.262	570	815000	1 129	510.	729000
5	2.542	1140.	1643000	2.201	990	1422000	1.968	880.	1272000
6	4.008	1800.	2591000	3.471	1560.	2244000	3 084	1380.	1993000

Diam.	$H_f = \frac{1}{6}L$			$H_f = \frac{1}{7}L$			$H_f = \frac{1}{8}L$		
in.	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0.0058	2 6	3740	0.0053	2 4	3460	0 0049	2 2	3170
⅝	0 0098	4.4	6340	0 0091	4.1	5900	0 0087	3.9	5620
¾	0 0156	7.0	10100	0 0145	6.5	9360	0 0136	6 1	8780
1	0 0321	14.4	20700	0.0299	13.4	19300	0 0279	12.5	18000
1¼	0 0561	25.2	36300	0 0519	23 3	33660	0 0486	21 8	31400
1½	0 0887	39.8	57300	0 0820	36 8	53000	0 0766	34 4	49500
2	0 1818	81 6	118000	0 1684	75.6	109000	0 1575	70.7	102000
2½	0 318	143.	205000	0 2941	132.	190000	0 2752	124.	178000
3	0 502	225	324000.	0.465	209.	300000	0 435	195.	281000
4	1.032	463	667000,	0 954	428.	616000	0 889	399.	575000
5	1.797	806.	1161000	1.664	750.	1075000	1.556	700.	1006000
6	2.834	1272.	1832000	2.625	1180.	1696000	2 455	1100.	1587000

Diam.	$H_f = \frac{1}{9}L$			$H_f = \frac{1}{10}L$			$H_f = \frac{1}{20}L$			$H_f = \frac{1}{40}L$		
in	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd	cfps	gpm	gpd
½	0 0047	2 1	3020	0 0045	2 0	2880	0.0027	1.2	1730	0.0018	0 8	1150
⅝	0 0080	3.6	5180	0 0078	3 5	5040	0 0049	2.2	3170	0 0033	1.5	2160
¾	0.0127	5.7	8210	0 0120	5 4	7780	0 0078	3.5	5040	0 0049	2.2	3170
1	0.0263	11.8	17000	0 0250	11.2	16100	0.0156	7.	10100	0 0111	5.	7200
1¼	0 0459	20 6	29700	0 0434	19 5	28100	0 0290	13.	18700	0.0201	9.	13000
1½	0 0724	32.5	46800	0 0686	30 8	44400	0.0468	21.	30200	0 0312	14	20200
2	0 1484	66 6	95900	0.1408	63 2	91000	0 1003	45	64800	0 0668	30.	43200
2½	0 2593	116.	168000	0.2460	110.	150000	0 1772	80.	115000	0 1203	54	77800
3	0.410	184	265000	0.389	175.	251000	0 2896	130.	187000	0.2005	90	130000
4	0.841	378.	544000	0.798	358.	516000						
5	1.467	660.	948000	1.392	620.	900000						
6	2.315	1040.	1496000	2.196	900.	1419000						

Table 183. Friction Loss, Hazen-Williams Formula

Leonard Metcalf, Cons. Engr., Boston,

Formula: $V = C_H R^{0.63} S^{0.54} (0.001)^{-0.04} = \dfrac{C_H R^{0.63} S^{0.54}}{(0.001)^{0.04}}$

$V = C_H(1.31826)\ {}^{0.63}S^{0.54} = C_H(0.55043) D^{0.63} S^{0.54}$

$\therefore\ V = 130(0.55043) D^{0.63} S^{0.54} = 71.5559 D^{0.63} S^{0.54}$

Diam., in		3		4		6		8		10		12	
		No.	Log.	No.	Log	No.	Log.	No.	Log.	No.	Log.	No.	Log.
Area of water cross-section, sq. ft. .		0 0491	8.6910	0.0873	8.9408	0 1963	9 2930	0 3491	9 5429	0.5454	9 7367	0 7854	9.8951
Contents, gals. per lin. ft.		0.3672	9.5649	0 6528	9 8148	1 4688	0 1670	2 6112	0 4168	4.0800	0.6107	5.8752	0.7690
Discharge, gals. per min,	$V = 1$	22 0	1 3431	39.1	1 5929	88.1	1 9451	156 7	2.1950	244 8	2.3888	352 5	2 5472
Discharge, million gals. per day, $V = 1$.		0.0317	4 5014	0.0564	4.7513	0.1269	5.1035	0.2256	5.3534	0.3525	5.5472	0.5076	5.705
Friction head, ft. per 1000 ft	$V = 1$.	1 85	0.2679	1 33	0 1221	0.82	9.9167	0 59	9.7709	0 455	9 6578	0.37	9 5655
	$V = 2$.	6.69	0.8253	4 78	0 6796	2.98	0.4741	2.13	0 3284	1 64	0 2153	1.33	0.1229
	$V = 3$.	14.17	1 1514	10 13	1 0057	6 31	0 8002	4.51	0.6545	3.48	0 5414	2 81	0 4490
	$V = 4$.	24.14	1 3828	17 26	1.2370	10 75	1 0316	7.69	0 8858	5.93	0 7728	4 79	0 6804
	$V = 5$.	36 50	1 5623	26 09	1 4165	16 26	1 2110	11 6	1 0653	8 96	0 9522	7 24	0 8599
	$V = 6$.	51.16	1 7089	36 57	1 5631	22 79	1 3577	16 3	1 2119	12 56	1 0989	10.15	1 0065
Discharge, gals. per min	$V = 3$.	66 1		117 6		264 3		470 0		734 0		1057 5	
Discharge, million gals. per day, $V = 3$.		0 0952		0 1692		0 3807		0 6768		1 0575		1.5228	

V = velocity, ft. per sec.
R = hydraulic radius, ft.
D = diam. of pipe in ft.
S = slope = ratio of head to length.

C_H = coefficient of roughness.
= 130 for ordinary cast-iron pipe.
= 140 for new cast-iron pipe.
= 100 for badly tuberculated iron pipe.
= 100 to 110 for steel pipe.

Constants and logs. — **Log.**

$(0.001)^{-0.04} = \dfrac{1}{0.75858} = 1.31826$		0.12000
130	130.00	2.11394
$(0.001)^{-0.04}(130) = 171.3734$		2.23394
$\left(\frac{1}{4}\right)^{0.63} = 0\ 417544$		9.62070
$1\ 31826 \times \left(\frac{1}{4}\right)^{0.63} = 0.55043$		9.74070
$130 \times 0.55043 = 71.5559$		1.85464
$C_H \times 1.31826 = 130 \times 1.31826 = 171.4-$		

Table 183. Friction Loss, Hazen-Williams Formula.—(*Continued*)

Diam., in		14		16		18		20		24		30	
		No.	Log.	No.	Log.	No.	Log.	No.	Log.	No.	Log.	No.	Log.
Area of water cross-section, sq. ft		1.069	0.029	1 396	0 1445	1.767	0.2473	2.181	0.3388	3.142	0.4971	4.909	0.6910
Contents, gals. per lin. ft ...		7.997	0.9029	10 445	1.0189	13 219	1.1212	16.320	1 2127	23 51	1,3711	36.720	1.5649
Discharge, gals. per min., $V = 1$		479 8	2 6811	626.7	2 7970	793.1	2.8994	979 2	2.9909	1,410	3 1492	2,203	3.3430
Discharge, million gals. per day, $V = 1$.		0.691	5.8394	0 902	5.9554	1.142	6 0577	1.410	6.1492	2.030	6.3076	3.173	6.5014
Friction head, ft. per 1000 ft.	$V = 1.$	0 31	9.4874	0.26	9.4197	0 229	9.3600	0 203	9 3066	0.164	9.2143	0.126	9.1012
	$V = 2.$	1 11	0 0448	0.95	9.9772	0 827	9.9175	0 731	9 8641	0 591	9.7717	0.0456	9.6587
	$V = 3$	2 35	0 3709	2 01	0.3033	1 752	0 2436	1 550	0 1902	1 253	0 0978	0 966	9 9848
	$V = 4.$	4 00	0 6023	3 42	0.5346	2 985	0.475	2 640	0 4215	2 134	0 3292	1 645	0 2161
	$V = 5.$	6 05	0 7818	5.18	0 7141	4 513	0 6544	3 991	0 6010	3 226	0 5087	2 487	0 3956
	$V = 6.$	8 48	0 9284	7 26	0 8607	6 325	0 8011	5 593	0.7477	4 522	0 6553	3 485	0 5422
Discharge, gals. per min., $V = 3.$		1439 0		1880 0		2379 0		2938 0		4230 0		6609 0	
Discharge, million gals. per day, $V = 3.$		2.073		2 707		3 426		4 230		6 091		9 518	

When $C_H = 100$ $V = 131.826R^{0.63}S^{0.54}$.
$C_H = 130$ $V = 171.373R^{0.63}S^{0.54}$.
$C_H = 140$ $V = 184.556R^{0.63}S^{0.54}$.

or when

$C_H = 100$ $V = 55.0D^{0.63}S^{0.54}$
$C_A = 130$ $V = 71.6D^{0.63}S^{0.54}$
$C_H = 140$ $V = 77.1D^{0.63}S^{0.54}$

Table 183. Friction Loss, Hazen-Williams Formula.—(*Concluded*)

Diam., in		36		42		48		54		60	
		No.	Log.	No.	Log.	No.	Log.	No.	Log.	No.	Log
Area of water cross-section, sq. ft.		7 069	0 8493	9 621	0 9832	12 566	1 0992	15 904	1 2015	19 63	1 2930
Contents, gals. per lin. ft. .		52 877	1 7233	71.971	1 8572	94 002	1 9731	118 972	2 0754	146 88	2 1670
Discharge, gals. per min, $V = 1$.		3,173	3 5014	4,318	3 6353	5,640 0	3 7513	7,138	3 8536	8,813	3 9451
Discharge, million gals. per day, $V = 1$.		4 568	6 6598	6 218	6 7937	8 122	6 9096	10 279	7 0120	12 690	7 1035
Friction head, ft. per 1000 ft.	$V = 1$.	0 102	9 0088	0 085	8 9307	0 073	8 8637	0 064	8.8034	0 056	8 7500
	$V = 2$.	0 368	9 5663	0 308	9 4882	0 263	9 4205	0 229	9.3608	0 203	9 3075
	$V = 3$.	0.781	9.8924	0 654	9.8143	0 558	9.7466	0 486	9 6869	0 430	9 6336
	$V = 4$	1.330	0 1238	1 111	0 0456	0 950	9.9780	0 829	9 9183	0 733	9 8649
	$V = 5$.	2 010	0 3032	1 679	0 2251	1 437	0 1575	1 253	0 0978	1 11	0 0444
	$V = 6$	2 817	0 4498	2 359	0 3717	2 014	0.3041	1 756	0 2444	1 55	0 1910
Discharge, gals. per min, $V = 3$.		9,518		12,955		16,920 0		21,415		26,438	
Discharge, million gals. per day, $V = 3$.		13 706		18 655		24 365		30 838		38 071	

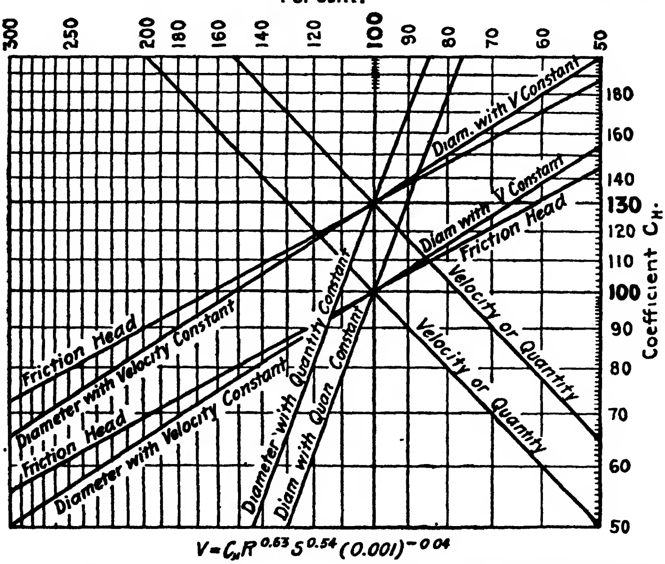
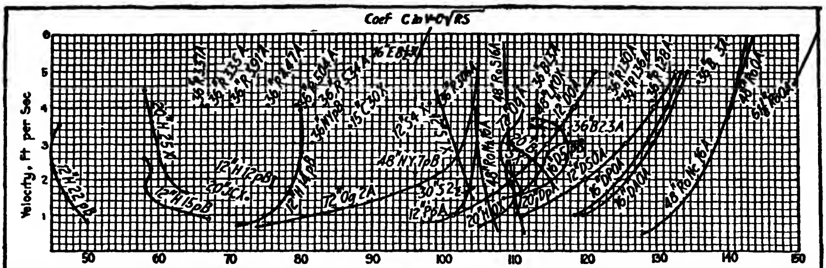


FIG. 317.—Rate of variation of certain factors in Hazen-Williams formula. (Leonard Metcalf, E. R., June 20, 1903.)

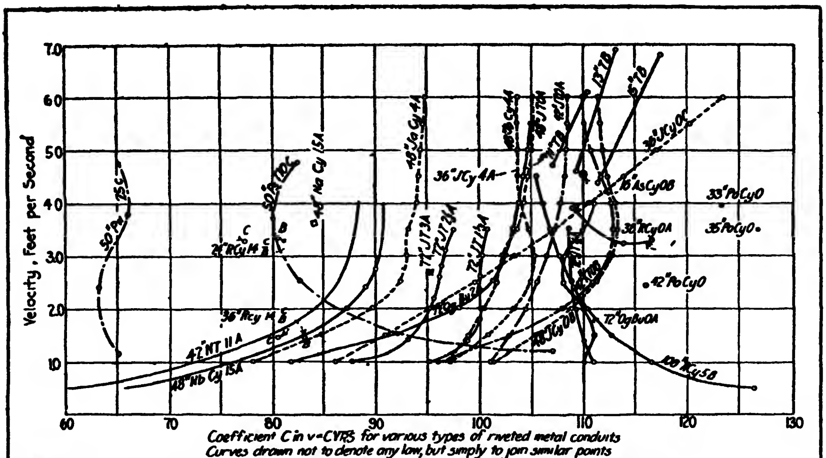
Flow Coefficients for a Large Cast-iron Main. Tests on the 30-in. and 36-in. Fisher Hill force main, Boston, laid in 1887, 5470 ft. long, with velocities from 2.52 to 2.64 ft. per sec., gave values of Chezy's C from 105 to 117,



MARK ON DIAGRAM	DIAM	LOCATION, COATING AND EXPERIMENTER	C	WT OF EXPT.	AGE YEARS	CONDITION	FIRST USED	SOURCE OF DATA	WEIGHT OF EXPERIMENT
72"Og A	72	Ogden, Utah (Steel all butt joints) Asphalt	108± to 111±	A	New		1897	Tr A S C E V40 p 471	
72"Og 2A	72	" " " " " " "	134 " 104.9	"	2	Not Examined	"	" V44 p 34	
61⅛"Ro OA	61⅛	Rosemary, Coal Tar,-Fitz Gerald	149.6 " 154.3	"	New		1898±	" p 88	
48"Ro OA	48	" " " Stearns	140.1 " 144.1	"	"		1878	" V35 p 259	
48"RoS16A	"	" S Pipe, Fitz Gerald	108.6 " 111.3	"	16	Tuberculated	"	" "	
48"RoNt16A	"	" N " "	104.4 " 107.8	"	"	"	"	" "	
48"RoNc16A	"	" " "	127.7 " 144	"	"	Cleaned	"	" "	
48"NY 7pB	"	N.Y. 10th Ave Uncoated, Duane	96	pB	7	Tuberculated	1873±	" V28	
48"LKOX	"	Loch Katrine, Asphalt, Jas. M. Gale	112.4	X	New	Good	1869	Hering's Kutter	A-Excellent
36"RL28A	36	Rochester, Tar,-Kuchling	129.5	A	1.28				B-Good
36"R130A	"	" " "	125.3	"	1.30				C-Bad
36"R136A	"	" " "	125.3	"	1.36				X=Unknown
36"R306A	"	" " "	99.2	"	3.06	Heavy			Quality
36"R335A	"	" " "	66.8	"	3.35	Growth			p=Probably
36"R337A	"	" " "	66.2	"	3.37	of	1894	Tr A S C E V44 p 56	
36"R391A	"	" " "	70.2	"	3.91	Sponge			
36"R447A	"	" " "	75.3	"	4.47	and			
36"R504A	"	" " "	79.8	"	5.04	Polyzoa			
36"R534A	"	" " "	81.9	"	5.34				
36"NYpB	"	N Y Croton, Kirkwood, 1867	81.6	pB		Heavily Tuberculated		Hering's Kutter	
30"S 2½X	30	Simpson	104	X	2 or 3			Tr. A S C E. V35 p 278	
20"B 5 X	20	Brush	109 to 115	"	5			" "	
20"JC X	"	Jersey City, Bailey, 1867 (See Fanning)	72.4	"		Tuberculated		Hering's Kutter	
20"H 25 X	"	Hamburg. Iben, 1876	58 to 64	"	25	Heavily incrusted		" "	
20"HOX	"	" Coal Tar "	105 " 110	"	New			" "	
20"Dp A	19⅔	Darcy, 1851	108.4 " 117.3	pA	"			" "	
16"D 5pB	16½	Dantzig. Asphalt C'T'd Lampe 1869-71	110.5 " 119.4	pB	5	Good		" "	
16"E 8½X	16	Edinburgh, W W	96 & 114	X	8 or 9			Tr. A S C E V35 p 278	
15"C 30 X	15	Crawley. Jas Leslie, 1855	86.9	"	30	Incrusted		Hering's Kutter	
12"S 7X	12	Simpson	99 & 104	"	Less than 7			Tr A S C E V35 p 278	
12"S 4X	"	"	98	"	" " 4			" "	
12"H12pB	"	Hamburgh. Iben 1876 541' long	75.7 & 76.2	pB	12	Slightly Tuberculated		Hering's Kutter	
12"H14pB	"	" " 1179' "	71 to 80	"	14	Slightly Incrusted		" "	
12"H15pB	"	" " 1808' "	58 " 67	"	15	Heavily Incrusted		" "	
12"H22pB	"	" " 1736 "	45 " 50	"	22	"		" "	
12"PpA	"	Paris. Darcy, 1851	96.9 " 104.8	pA	Old	Carefully cleaned		" "	
16"DAOA	16	Detroit, Coal Tar Army Tangent; Williams Hubbell Fenkell	119.3 " 134.5	A	New	Good	1898	Tr A S C E V47 p 159	
16"DPOA	"	" " Porter "	118.3 " 133.6	"	"	"		" "	
12"DSOA	12	" " So Street "	111.1 " 132.9	"	"	"	1899	" V47 p 181	
12"DOA	"	" " Straight Pipe "	101.5 " 121.3	"	"	"	"	" " p 187	
36"B.3A	36	Boston " Sherman	136	"	0.3±		1894	" V44	
36"B1.3A	"	" "	113	"	1.3±		"	" "	
36"B2.3A	"	" "	114	"	2.3±		"	" "	

FIG. 318.—Values of Chezy's C for new and tuberculated cast-iron pipes. Compiled by Engineering Bureau, Board of Water Supply, New York.

averaging 111; Kutter's $n = 0.0124$. Values are approximate only, being incidental to engine test. When pipe was 10 yrs. old tests by C. W. Sherman on a 5154-ft. length gave $C = 103$, and $n = 0.0133$. For a 36-in. main laid in



Mark on Diagram	Dia	Location	Material	Joint	Age Yrs.	Wt of Expt.	First Used	Source of Data	
72 JT 1/6 A	72"	Jersey City Water Supply Co	Steel	Taper	1/6	A	Jun. 1903	Official Records	
72 JT 1 1/3 A	"	"	"	"	1 1/3	"	"	" "	
72 JT 2 1/3 A	"	"	"	"	2 1/3	"	"	" "	
▣	"	"	"	"	3	"	"	" "	
48 JT 0 A	48	Cond. 2 E. Jersey W. Co.* abv. Pompton Notch	"	"	New	"	May 1896	Pp 33 53 Herschel's 115 Exps	
48 J Cy 0 B	"	" 1 " "	"	Cylinder	"	B	Apr 1892	Pp 26 52 " "	
48 Ja Cy 4 A	"	" 1 " " " " "	"	"	4	A	"	" " "	
48 Jb Cy 4 A	"	" 1 " " below " "	"	"	"	"	"	" " "	
42 JT 0 A	42	" 2 " " " " "	"	Taper	New	"	Aug. 1896	Pp. 33,53 " "	
42 KT 0 B+	"	Kearney Exten E Jersey Water Co.	"	"	"	B+	Jan. 1896	" " "	
36 J Cy 0 C	36	E Jersey Water Co.	"	Cylinder	"	C	Feb. 1892	Pp 32,52 " "	
△	"	" " " "	"	"	4	A	"	Pp. 52 " "	
36 R Cy 14 C/B	"	Rochester N Y	W Iron	"	14	C/B	1876	Pp. 30,34,54 " "	
50 B T 10 C	50	Brilliant Pump Sta. to Highland Res. 1 Pitts.	Steel	Taper	10	C	Early part '94	Official Records	
50 P 25 C	"	" " " " " " "	W Iron		25	"	1879	" "	
108 H Cy 5 B	108	Holyoke, Mass.	"	Cylinder	5	B	1882±	Pp. 29,34,54 Herschels Exps	
24 R Cy 14 C/B	24	Rochester N.Y.	"	"	14	C/B	1876	Pp. 30,34,54 " "	
38 R Cy 0 A	38	" "	Steel	"	New	A	Aug 1894	" " " " " "	See Tran. A.S.C.E. Vol.
11 TB	11			Taper	Good as New	B		Pp. 33,53 " "	40 P 540 for Corr.
13 TB	13			"	"	"		" " " " "	
15 TB	15			"	"	"		" " " " "	
●	16	Astoria, Oregon		Cylinder	New	"		Pp. 34,53 " "	
72 Og B 0 A	72	Ogden, Utah, Pioneer Elec Pow. Co.	Steel	Butt	"	A	1897	Tran. Am. Soc. C.E. Vol. 40 P 471	
33 Po Cy 0	33	Portland, (Ore.) W.W.	"	Cylinder	"	?		" " " " " 26 P 203	On length of 34,176'
35 Po Cy 0	35	" " "	"	"	"	"		" " " " " " "	" " " 39,009'
42 Po Cy 0	42	" " "	"	"	"	"		" " " " " " "	" " " 50,965'
17 TC 1	17	Texas Creek Cal C=131.1 for V=20.14	Sheet Iron		1	"		Ham Smith Hyd P 311	" " " 4,430'
72 Og B 2 A	72	Ogden, Utah, Pioneer Pow Co	Steel	Butt	2	A		Tran. Am. Soc. C.E. Vol. 44 P 311	
48 N Cy 15 A	48	Cond 1 Newark, above Pompton Notch	"	Cylinder	15	"	Apr 1892	T. Merriman	" " " 23,871'
48 N Cy 15 A	"	" 1 " below " "	"	"	"	"	"	"	
42 N T 11 A	42	" 2 " " " "	"	Taper	11	"	Aug 1896	"	

A:- Denotes excellent experiments
B:- " good
C:- " poor

†Only 8 yrs since cleaning and painting

*E. Jersey Water Co's conduits became property of the City of Newark about 1897

Fig. 319.—Values of Chezy's C for various types of riveted metal conduits. Compiled by Engineering Bureau, Board of Water Supply, New York.

1894, incidental to an engine test 1 yr. later, $C = 136.6$ for $V = 4.7$; when the pipe was 2 yrs. old, the following values of coefficients were obtained:

V ft. per sec.	1.1	1.5	2.0	2.2	2.6	3.1	3.5	3.7	4.3	4.5
n.....	0.0116	0.0121	0.0123	0.0125	0.0125	0.0127	0.0127	0.0127	0.0126	0 0127
C.....	117.0	114.9	114.9	113.3	113.7	113.5	113.7	113.7	113.0	1 14.0

Experiments in 1897, with $V = 3.2$, gave $C = 114$, $n = 0.0126$.

Flow in Spiral Riveted Pipe. Experiments by E. W. Schoder at Cornell showed the carrying capacity for 4-in. and 6-in. spiral riveted pipe, to be 6 per cent. in excess of new cast-iron pipe, of the same nominal diam.; perhaps explainable by the fact that spiral pipe is made full diam.

FLOW IN WOOD PIPE

Formulas. From experiments by E. A. Moritz (Trans. Am. Soc. C. E., Vol. 74, Oct., 1911) on the flow in wood pipes, R. G. Dieck concludes (1) Kutter's n for wood stave pipe, continuous or jointed, increases with a decrease in velocity (varies inversely with some power of the velocity); for practical purposes its value lies between 0.0100 and 0.0110. (2) There is no great necessity for altering the Kutter formula.

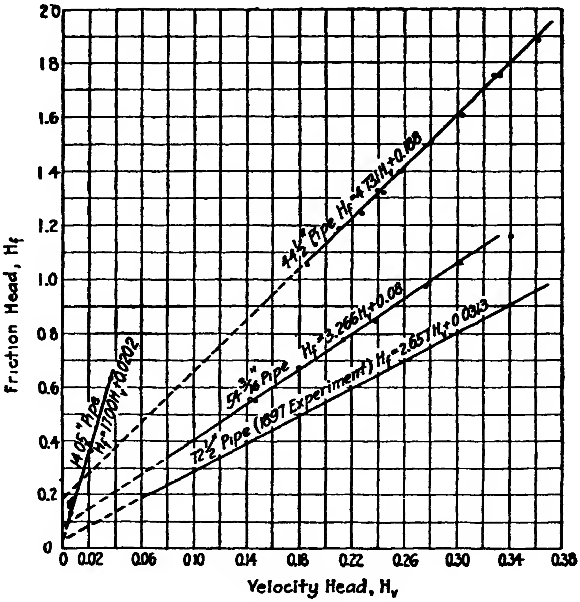


Fig. 320.
(T. A. S. C. E., Vol. 49, 1902, p. 133)

Noble's Experiments. (Trans. Am. Soc. C. E., Vol. 49, 1902, p. 163.) Author's conclusions: (1) Within the range of these experiments, from 14-in. to 72-in. pipes, and between friction heads of 0.2 to 2 ft. per thousand, the friction head, H_f, varies with the square of the velocity. (2) The velocity varies as some odd exponent of the diam. In three out of four experiments, this exponent is very nearly $\frac{1}{1.75}$. (3) The formula which conforms very closely to three out of four experiments is $V = 1.76\, D^{\frac{1}{1.75}} \sqrt{H_f \pm b}$.* (4) This formula will probably give results within 2 per cent. of true values.

* b is a constant of small value, probably due to causes mentioned by Prof. Merriman: The quantity, b, as a tentative hypothesis, is the correction to be applied to the observed friction head, H_f, to give the true friction head, $H_f - b$, which corresponds to the mean velocity, V. The probable values of b may be computed from, $b = 0.009\, C_w^{\,3} D H_f$.

From Fig. 320 it will be seen that the velocity head, as compared with the friction head, conforms very closely to a straight line. The evidence is very strong that a formula on this hypothesis is correct, as it most nearly fits the experiments so far conducted. If H_f = friction head in feet per 1000 ft., H_v = velocity head = $\frac{V^2}{2g}$, V = velocity in feet per sec., values of H_f and V for the different sizes of pipe become:*

Pipe diam., in.	H_f	V
14	$17.00\ H_v + 0\ 0202$	$1.9463\sqrt{H_f - 0.0202}$
44	$4.731\ H_v + 0\ 188$	$3\ 6895\sqrt{H_f - 0.188}$
54	$3.266\ H_v + 0.08$	$4\ 4405\sqrt{H_f - 0.08}$
72	$2.657\ H_v + 0.0313$	$4.9232\sqrt{H_f - 0.0313}$

$V = C_w\sqrt{D(H_f - b)}$ or, using hydraulic radius, R,
$= C_R\sqrt{R(H_f - b)}$, in which $C_R = 2C_w$ as tabulated below.

Table 184. Coefficient in Theron A. Noble's Formula for Flow in Wood Pipes

Size pipe, in	C_w	b	Size pipe, in	C_w	b
14	1.80	0.031	48	1.98	0.146
18	1.82	0.041	54	2.09	0.080
24	1.84	0.073	60	2.06	0.063
30	1 86	0.114	66	2.03	0.047
36	1 88	0.145	72	2.00	0.031
42	1.91	0.176			

Values of C_w and b are for pipes with smooth interiors; further factor of safety should be allowed, to cover increased loss of head, when pipe may be covered with growth that may retard flow.

Table 185. Data Regarding Loss of Head in Wood Pipes

Trans. Am. Soc. C. E., 1898, Vol. 40, p. 512

Locality	Diam. in.	Vel. ft. per sec.	Chezy's C	Kutter's n	Experimenter
West Los Angeles Water Co.	14	0.7 1.2 1.5	102 0 111 0 112.0	0.011 0.011 0.011	A. L. Adams
Astoria, Ore..........	18	3.605	132.9	0.010	A. L. Adams
Butte City, Mont........	24		120.0 130.0	0.010	F. B. Gutillius
Denver, Colo.	30	2.33	140.0 150.0	0.0096	J. D. Schuyler
Ogden, Utah..........	72	0.53 1.48 2 69 3.65	69.0 115.0 120.0 123.0	0.012 0.015	Marx, Wing and Hoskins

The simplest formula, as well as the most exact for such pipes, is of the form, $H_f = mV^n$, where H_f is the loss of head, and V the velocity; m is obtained

* T. A. S. C. E., Vol. 49, 1902, p. 136.

directly from the logarithmic plotting by noticing the intercept on the line for $V = 1$; n is equal to the slope of the line drawn through the plotted points.

Table 186. Values of H_f for 4 Pipes in Fig. 320

(Derived by log. plotting of observations)

Diam., in.	H_f, per 1000 ft.
14	$0.300\ V^{1.73}$
44	$0.125\ V^{1.73}$
54	$0.0815\ V^{1.73}$
72 (1897)	$0.062\ V^{1.73}$
72 (1899)	$0.048\ V^{1.94}$
2.09 (seamless drawn brass pipe)	$2.62\ V^{1.759}$

(E. W. Schoder, T. A. S. C. E., Vol. 49, 1902, p. 148.)

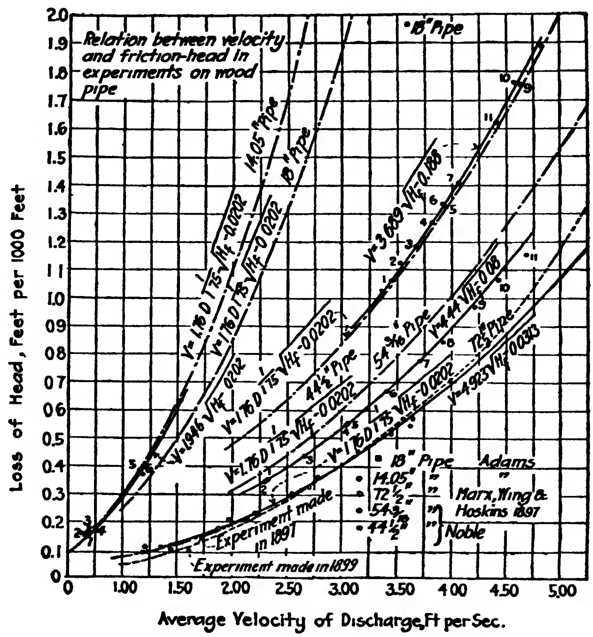


FIG. 321.—Relation between velocity and friction head in experiments on wood stave pipe.

Solid lines are plotted from tabulated formulas, p. 572; dotted lines, from $V = 1.76\ D^{1.75}\sqrt{H_f \pm b}$, p 571. Exact diameters of pipes are given

Hydraulic Tables Based on Kutter's Formula. (Pacific Coast Pipe Co.) In Table 188, $C\sqrt{R}$ and $AC\sqrt{R}$ were derived by assigning values to n agreeing closely with various experiments on wood pipe, varying from 0.008 for the smallest size to 0.014 for the largest; corresponding $C = 123$ to 97; and $C_H = 100$ to 140. 1. Given diam. 2 ft. 8 in., length 4000 ft., and fall 300 ft., to find the velocity and discharge. Slope = 300 in 4000 = 1 in 13.3. Substi-

tuting in the formula: $V = C\sqrt{R} \times \sqrt{s}$, the value $C\sqrt{R}$ equals 98.34 (Table 188, opposite 2 ft. 8 in.) and $\sqrt{s} = 0.2752$ (Table 187, opposite slope, 1 in 13.2), velocity = 27.06 ft. per sec. For computing discharge, use the formula: $Q = AV$, now that V is known, or the formula, $Q = AC\sqrt{R}\sqrt{s}$. In Table 188, we find that for diam. = 2 ft. 8 in., $A = 5.585$; $AC\sqrt{R} = 549$. 549 ×

Table 187. Values of $\sqrt{s}$ for Given Slopes

s = Head ÷ Distance

Fall, ft. per mi.	Slope, 1 in.	$\sqrt{s}$	Fall, ft. per mi.	Slope, 1 in.	$\sqrt{s}$
1	5280.0	0.0137	43	122.8	0.0902
2	2640.0	0.0194	44	120.0	0.0912
3	1760.0	0.0238	45	117.3	0.0923
4	1320.0	0.0275	46	114.8	0.0933
5	1056.0	0.0307	47	112.3	0.0943
6	880.0	0.0337	48	110.0	0.0953
7	754.3	0.0364	49	107.7	0.0963
8	660.0	0.0389	50	105.6	0.0973
9	586.6	0.0412	51	103.5	0.0982
10	528.0	0.0435	52	101.5	0.0992
11	443 6	0.0456	53	99.60	0.1001
12	440 0	0.0476	54	97.77	0 1011
13	406.1	0.0496	55	96.00	0.1020
14	377.1	0.0514	56	94.28	0.1029
15	352.0	0.0533	57	92.63	0.1039
16	330.0	0.0550	58	91 03	0.1048
17	310.6	0.0567	59	89.49	0 1057
18	293.3	0:0583	60	88.00	0.1066
19	277.9	0.0599	65	81 23	0.1109
20	264.0	0.0615	70	75.43	0.1151
21	251.4	0.0630	80	66 00	0.1230
22	240.0	0.0645	90	58.66	0 1305
23	229.6	0.0660	100	52 80	0 1376
24	220.0	0.0674	120	44.00	0 1507
25	211.2	0.0688	140	37.71	0 1628
26	203.1	0.0701	160	33.00	0 1740
27	195.2	0.0715	180	29.33	0 1846
28	188.6	0.0728	200	26.40	0.1946
29	182.1	0.0741	240	22 00	0.2132
30	176.0	0.0753	280	18.86	0 2302
31	170.3	0.0766	320	16.50	0.2461
32	165.0	0.0778	360	14.66	0.2611
33	160.0	0.0790	400	13.20	0.2752
34	155.3	0.0802	450	11.73	0.2919
35	150.9	0.0814	500	10.56	0.3077
36	146.6	0.0825	600	8.800	0.3371
37	142.7	0.0837	700	7.543	0.3641
38	139.0	0.0848	800	6.660	0.3892
39	135.4	0.0859	900	5.866	0.4128
40	132.0	0.0870	1000	5.280	0.4351
41	128.8	0.0881	1500	3.520	0.5329
42	125.7	0.0891			

L. M. Grant for Pacific Coast Pipe Co.

0.2752 gives a discharge of 151.1 cu. ft. per sec. 2. Given diam. (8 in.), length (1000 ft.) and discharge (3000 cu. ft. per hr.). What is theoretical head? 3000 cu. ft. per hr. = 0.833 cu. ft. per sec. From Table 188, opposite 8 in.,

Table 188. Values of A and R, $C\sqrt{R}$ and $AC\sqrt{R}$ for Wood Stave Pipe

Diam., ft.	in.	R = Hydraulic radius, ft.	A = Area, sq. ft.	$C\sqrt{R}$	$AC\sqrt{R}$
0	2	0.0417	0.0218	20.16	0.439
	3	0.063	0.0491	25 81	1.267
	4	0.083	0.0873	31 85	2.780
	6	0.125	0.196	40 76	7.99
	8	0.167	0.349	48.49	16.92
	10	0.208	0.545	54.74	29.83
1	0	0.250	0.785	59 71	46.87
	2	0.292	1.069	65 19	69.69
	4	0.333	1.396	69 22	96.63
	6	0.375	1.767	73.67	130.2
	8	0.417	2.182	77.77	169.7
	10	0.458	2.640	82 06	217.
2	0	0 500	3.142	87 40	275.
	2	0.542	3 687	90 56	334.
	4	0 583	4.276	93 35	399.
	6	0.625	4 909	95.89	470.
	8	0 667	5.585	98.34	549.
	10	0 708	6 305	101.4	639.
3	0	0.750	7.068	104.7	740.
	2	0.792	7.875	108.8	857.
	4	0 833	8 726	111.4	972.
	6	0.875	9 621	114 2	1100.
	8	0.917	10 559	116.7	1230.
	10	0 958	11 541	119.2	1380.
4	0	1.000	12 566	121.7	1530.
	2	1.042	13 635	123 9	1690.
	4	1.083	14 748	126 1	1860.
	6	1.125	15.904	128.3	2040.
	8	1.167	17 104	130.4	2230.
	10	1.208	18.348	132.5	2430.
5	0	1.250	19.635	135.6	2660.
5	6	1.375	23 758	142 7	3390.
6	0	1.500	28 274	149.2	4220.
6	6	1.625	33.183	154.5	5130.
7	0	1.750	38.485	160 0	6160.
7	6	1.875	44.179	164.8	7280.
8	0	2.000	50.266	170.7	8580.
8	6	2.125	56.745	176.1	9990.
9	0	2.250	63.617	181 6	11600.
9	6	2.375	70.882	186.7	13200.
10	0	2.500	78.540	191.6	15000.
10	6	2.625	86 590	196.3	17000.
11	0	2.750	95.033	202.5	19200.
11	6	2.875	103.87	207.9	21600.
12	0	3.000	113.10	213.7	24200.

$AC\sqrt{R} = 16.92$. Substituting in the formula, $\sqrt{s} = \frac{Q}{AC\sqrt{R}}$, $\sqrt{s} = 0.0492$. Referring to Table 187, the value nearest this is 0.0496, corresponding to a fall of 1 in 406. By proportion, 1:406 :: required head : 1000. Head = 2.46.
3. Given the available head = 250 ft., distance from the source to point of delivery = 11,000 ft. (length), and the required discharge, 10 cu. ft. per sec. What must be the diam.? $s = 1/44$. The corresponding $\sqrt{s}$ (Table 187) = 0.1507. Then $AC\sqrt{R} = Q/\sqrt{s} = 66.35$. Referring to Table 188, the nearest value is 69.69, opposite 1 ft. 2 in. A 14-in. pipe will therefore be required.

FRICTION-HEAD DIAGRAMS*

(I. P. Church)

Effect of Age. With increasing age of service, cast-iron pipes become corroded and tuberculated (if originally tar-coated, this action may be much retarded) which diminishes the discharge under the same head (both from increased roughness and diminished sectional area). E. B. Weston recommends that friction head for a given Q be taken as 16 per cent. greater than when the pipe is new and clean, for each 5 yrs. of age; that is, for an age of 15 yrs., take as friction head the value obtained by multiplying the result given by the diagram by 1.48.

Branching Pipes Problem. The reservoir R supplies R' and R''. The three cylindrical pipes have a junction at J (Fig. 325). When flow is steady Q (cu. ft. per sec. in main pipe) = sum of Q' and Q'' in the other pipes. H_F, H'_F and H''_F are the respective friction heads. The vertical drop AB is equal to $C_o\frac{V^2}{2g}$† $+ \frac{V^2}{2g}$; and similarly the drop $CD' = C_e$‡$\frac{V'^2}{2g} + (\frac{V'^2}{2g} - \frac{V^2}{2g})$; and $CD'' = C_e'\frac{V''^2}{2g} + (\frac{V''^2}{2g} - \frac{V^2}{2g})$. The depths $U'N'$ and $U''N''$ at which the pipes discharge under water are immaterial. There are no nozzles at N' and N'', that is, the jets have the same sectional areas and velocities as the water in the respective pipes. In practice, the velocities in a system of pipes are rarely over 10 ft. per sec., and the pipes are quite long, so that, it is sufficiently accurate to neglect the small drops AB, CD' and CD'', and consider that the whole drop from the surface of R to that of R' = sum of the friction heads H_F and H'_F; also the whole drop from R to $R'' = H_F + H''_F$.

Example 1. Determine the proper diam. for the 3 pipes, given $Q' = 13$ and $Q'' = 20$ cu. ft. per sec. (so that Q must equal 33); $L = 30{,}000$ ft., $L' = 10{,}000$ ft., and $L'' = 15{,}000$ ft. The difference of elevation, $R - R' = 60$ ft.; $R - R'' = 85$ ft. Solution for clean cast-iron pipe: Assume one diam., or the friction head H_F of the main pipe; say the latter. Take $H_F = 40$ ft. A steady flow is to take place of 33 cu. ft. per sec., and the friction head is to be at the rate of $\frac{40}{30} = 1.33$ ft. per thousand ft. In Diagram 323, p. 578, the vertical line for 1.33 (interpolating) intersects the horizon-

* For instructions on the construction of a logarithmic diagram covering the general case, see Eng. Rec., Sept. 3, 1904.
† C_o = coefficient of entry loss.
‡ C_e and C'_e = coefficient for loss of head due to elbows, sudden change of section, etc.

tal lines for $Q = 33$, in a point corresponding to a diam. of 38 in. (among the lines sloping up to the right), while among the lines sloping down to the right we find the velocity in a 38-in. pipe would be 4.1 ft. per sec. (not ex-

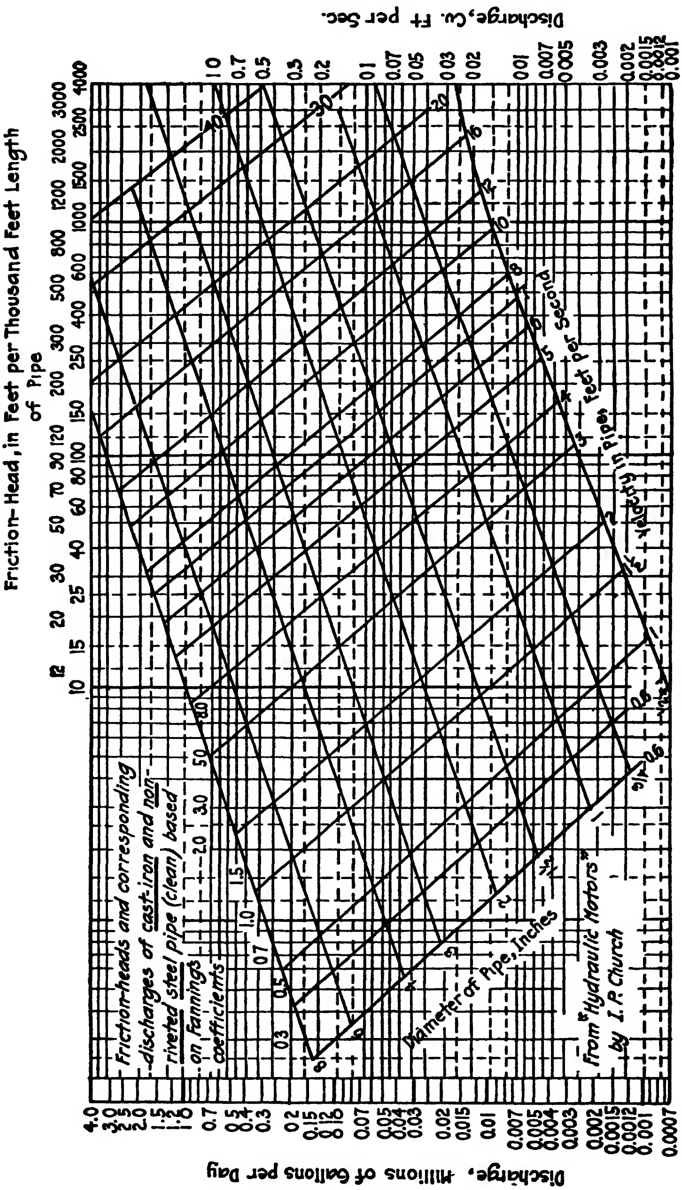


FIG. 322.—½ in. to 8 in. clean cast-iron pipes.
(Larger pipes, Fig. 323, p 578)

treme). Deducting the assumed H_F from 60 ft., the corresponding value of $H'_F = 20$ ft., *i.e.*, at the rate of 2 ft. per thousand. From the same diagram,

the intersection of the vertical 2 with the horizontal line for $Q = 13$, is a point on the line for a 25-in. pipe; in which the velocity would be 3.8 ft. per.

Prof. I. P. Church.

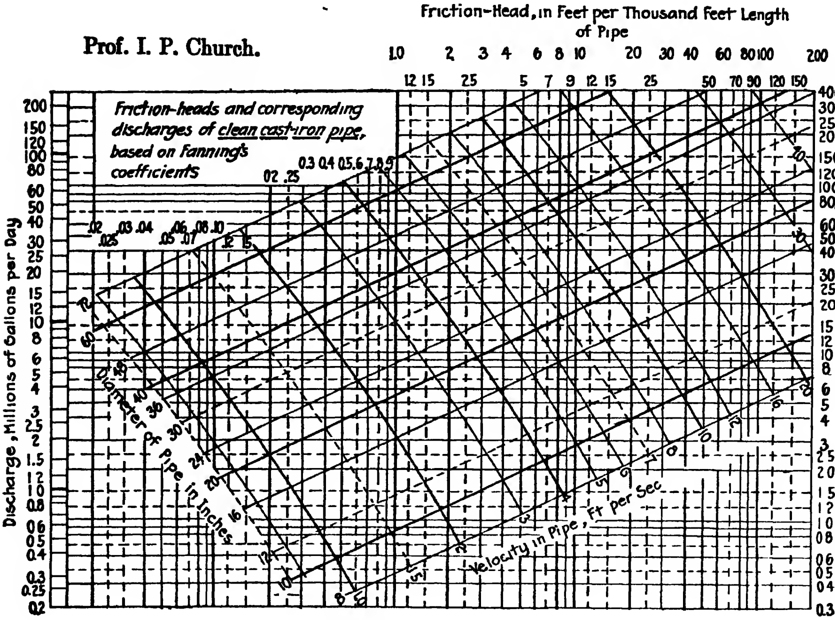


FIG. 323.—8 in. to 72 in. clean cast-iron pipes.
(Smaller pipes, Fig. 322, p 577)

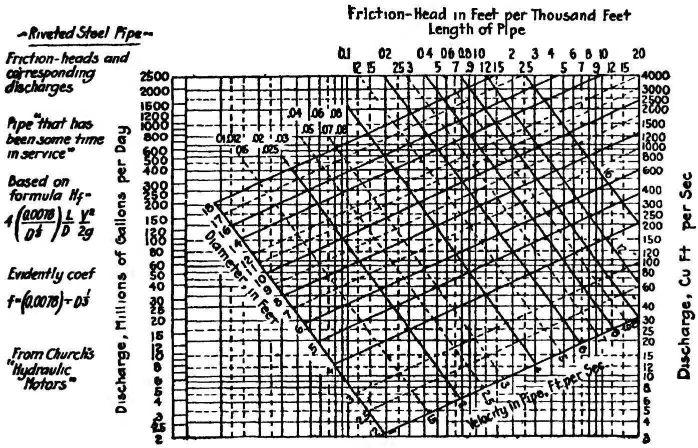


FIG. 324.—Riveted steel pipes.

sec. (a permissible value). Similarly deducting H_F from 85 ft., $H''_F = 45$ ft., which is at the rate of $\frac{45}{15} = 3$ ft. friction head per thousand. With $Q = 20$, the diagram gives 27.5-in. pipe, with a velocity = 5.0 ft. per sec. If H_F had been assumed somewhat greater than 40 ft., a smaller diam.

would have resulted for the main pipe, with a higher velocity than before; but larger diam. and smaller velocities in the branch pipes. Results should be sought involving least costs, with sufficient velocities (above 2 ft. per sec.) to prevent deposit of silt.

Example. 2. In the above example the diameters of the pipes were sought; but if they were given, and rates of flow to be determined, assume a trial value for Q and find from Diagram 323, the friction head per thousand feet of pipe of the given diam., thence the value of H_F, for the actual length, $EJ = L$. Values of H'_F and H''_F corresponding to H_F are now noted, and corresponding values of Q' and Q'' found from the diagram. $Q' + Q''$ should equal Q. If not, as the result of the first trial, assume a new value for Q; and so on, until the necessary equality is obtained. In the preceding paragraph it is assumed that water flows into R' and R'', and out of R, but if R' is at sufficient elevation, or if the pipe EJ is small, water may flow out of R', as well as out of R. In such case, the summit C would be lower than the surface of R', and $Q + Q' = Q''$. Similar principles and methods apply to any system or network of pipes.

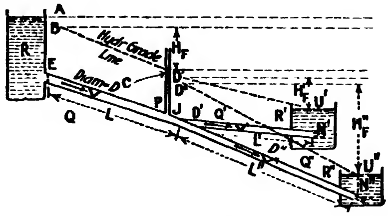


FIG. 325.

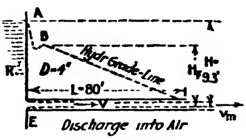


FIG. 326.

Single Cylindrical Clean Iron Pipe. No Nozzle, so that V_m, velocity of jet, $= V$, velocity in pipe. Given L, D, and H, find V and Q. For a pipe of this (relative) length, H_F will probably take up a large part of the whole head, H. (Fig. 326). Assume, a trial value of 7 ft. for H_F. This is at the rate of $\left(\frac{7}{0.080}\right) = 87.5$ ft. friction head per thousand. From Diagram 322, $V = 9.1$ ft. per sec. and $\frac{V_m^2}{2g}$ is 1.3 ft.; so that $\frac{{}^*C_oV^2}{2g} = \frac{1}{2} \times 1.3 = 0.65$ ft. To realize the trial value, 7 ft., for H_F, the whole head H would need to be $0.65 + 7 + 1.3 = 8.95$ ft., but this lacks 0.35 ft. of 9.3 ft., the available head. For the next trial, it will probably occasion no great error if the whole of this 0.35 be added to 7 ft. That is, assume $H_F = 7.35$ ft. $\left(\frac{7.35}{0.080}\right) = 92$ ft. friction head per thousand feet. Diag. 322 gives $V = 9.4$ ft. per sec. and $\frac{V^2}{2g} = 1.4$ ft.; $\frac{1}{2} \times 1.4 = 0.7$ ft. Adding, $0.7 + 7.35 + 1.4 = 9.45$ ft., which is so near 9.3 ft. that the second trial may be considered final. Hence, $V = V_m = 9.4$ ft. per sec. and Q (from diagram) $= 0.81$ cu. ft. per sec. N.B. If the jet discharges under water, the results are the same provided the surface of the water in the receiving reservoir is 9.3 ft. below that in the supply reservoir, R. (Both reservoirs large.)

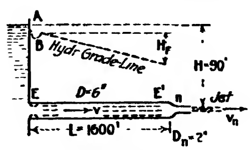


FIG. 327.

* C_o = coefficient of entry loss

Single Cylindrical Clean Iron Pipe with Nozzle. The jet Fig. 327 discharges into the air with velocity V_n, to be found. The diam. of the jet is 2 in.; of pipe, 6 in. Solution: V, in pipe, is only $\frac{1}{9}$ of V_n, since the sectional areas are in this (inverse) ratio. From Bernoulli's theorem, $H = {}^*C_o\frac{V^2}{2g} + H_F + C_e\frac{V_n^2}{2g} + \frac{V_n^2}{2g}$, where $H = 90$ ft. $C_o = 0.50$; $C_e = 0.05$ (rounded nozzles, or conical play pipe). H is made up of 4 terms, or heads, of which friction head H_F in the pipe is not necessarily a large item, because V is small compared with V_n. Solve by Diagram 322 with successive trial values of some unknown quantity, say V. First assume $V = 5$ ft. per sec., for which friction head would be at the rate of 18 ft. per 1000 ft.; hence, $H_F = \frac{1600}{1000} \times 18 = 28.8$ ft. Since $V = 5$, and $V_n = 9 \times 5 = 45$, $\frac{V^2}{2g} = 0.40$ ft. and $\frac{V_n^2}{2g} = 31.4$ ft. Hence, the two small losses of head would be $\frac{1}{2} \times 0.4 = 0.2$; and $\frac{1}{20} \times 31.4 = 1.57$ ft. $0.2 + 28.8 + 1.57 + 31.4 = 61.97$ ft. (but it should be 90 ft.). Second trial: Take $V = 6$ ft. per sec., V_n would be 54 ft. per sec., and the velocity heads 0.56 and 45 ft. respectively; hence, the two small losses of head, 0.28 and 2.25 ft. Now 6 ft. per sec. in a 6-in. pipe implies a friction head at the rate of 25 ft. per 1000 ft. Therefore, $H_F = \frac{1600}{1000} \times 25 = 40$ ft. $0.28 + 40 + 2.25 + 45 = 87.53$ ft., the difference between this and 90 ft. is so small that 6.1 ft. per sec. may be considered final for V; from which follow 55 ft. per sec. for V_n, and 1.2 cu. ft. per sec. for Q. With increasing age the discharge and V would gradually diminish unless the pipe were kept clean. If the entrance E were rounded slightly, a slight increase in Q would result.

Instead of diam. being given, what should be its value that 80 ft. of the total 90 ft. may be available to produce the jet velocity, V_n? 80 ft. $= \frac{V_n^2}{2g}$. $V_n = 71.8$ ft. per sec. At E' the loss of head would be $\frac{1}{20}$ of $80 = 4$ ft., while that at E may be neglected. This leaves 6 ft. for H_F, which is at the rate of 3.75 ft. per 1000 ft. A 2-in. jet at 71.8 ft. per sec. is discharging 1.56 cu. ft. per sec. From Diagram 323 a diam. of 9.9, say 10 in., must be given to the pipe in Fig. 327. This change of design calls for a greater consumption of water (1.56 instead of 1.20) but the "Kinetic Power" (*i.e.*, the kinetic energy of the jet)† $\left(\frac{Q\gamma}{g}\right) \times \left(\frac{V_n^2}{2}\right)$ will be more than doubled: 14.2 horsepower instead of 6.4.

Supply to Turbine or Discharge from Centrifugal Pump. Consider a turbine supplied through 24-in. riveted pipe, 2000 ft.; suction-head, 10 ft.; whole head, 80 ft.; consumption of water in steady flow limited to 20 cu. ft. per sec. From Diagram 323, $V = 6.4$ ft. per sec. and friction head $= 5.8$ ft. per thousand, $11.6 \times \frac{136^‡}{100} = 15.8$ ft., drop in hydraulic grade line, while $(V^2 \div 2g)\ (1 + 0.5) = 1.02$ ft. $=$ loss at entrance of pipe. Hence piezometer hight at entrance to turbine is $70 - (15.8 + 1.02) = 53.18$ ft.; and vertical distance to tail-

* C_o = coefficient of entry loss.
† γ = weight of water, lbs. per cu. ft.
‡ See top of p. 557.

surface = 63.18 ft. In computing efficiency of turbine, 63.18 + ($V^2 \div 2g$), *i.e.*, 63.86 ft. must be used, and not 80 ft., since pipe is not part of turbine. If diam. 24 in. be adopted for supply pipe, loss of head thereby is about 16 ft. and loss of power = 20 × 62.5 × 16 ft.-lbs. per sec. or 36.4 hp.; loss of head in pipe is $\frac{1}{5}$ of total head of mill site. For 36-in. pipe, loss of head is only 2 ft.; however, interest on extra cost of pipe might be greater than income from power due to head saved.

If, in expression for friction head in long pipe, (p. 556) $Q \div \left(\frac{\pi D^2}{4}\right)$, be substituted for V,

$H_f = \frac{32\, f\, L}{\pi^2 g} \cdot \frac{Q^2}{D^5}$; from which, if f be considered constant (as rough approximation), friction head is inversely proportional to fifth power of diam. Power lost in supply pipe =

$$P_f = Q\gamma H_f{}^* = \frac{32 f L}{\pi^2 g}\gamma \cdot \frac{Q^3}{D^5}$$

Whence, approximately, power lost in supply pipe is directly proportional to cube of flow and inversely to fifth power of diam. Doubling discharge without change in length or diam., would involve about 8 times as much loss of power. If, in preceding example, centrifugal pump were substituted for turbine, pumping 20 cu. ft. of water per sec. from tail race to reservoir, summit of piezometer column at pump would stand at hight above pump equal to hight of reservoir surface above pump + H_f; or, for 24-in. pipe, 70 + 15.8 = 85.8 ft.; and therefore 15.8 ft. above reservoir level.—("Hydraulic Motors," I. P. Church, 1908.)

SPECIAL CASES OF FLOW

Time of Filling a Water Main. Method of computation is shown by an example and is approximate only. Fig. 328 is the profile of a 48-in. pipe line.

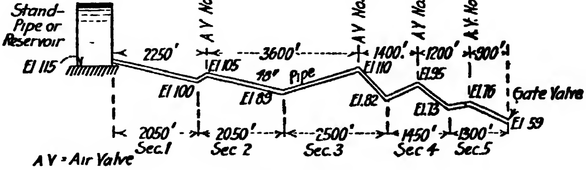


FIG. 328.

First air valve controls about 2250 ft. of pipe, and so on, as shown. When water is admitted at standpipe it first fills nearest depression, then rises until it overflows next summit. Hence, air displaced at first will probably discharge through all four valves. Until second depression is filled, air will escape at standpipe and through at least the first two air valves. After this, air from section 2 alone will find egress from the first air valve. Now, as water rises in sections 2 and 3, corresponding volumes of air will be expelled at valves

* γ = weight of water, lbs per cu ft.

1 and 2. After water overtops the second summit, air displaced from sections 3 and 4 will escape from valves 2 and 3, at least; and possibly some from valve 4. After third depression has been filled and water has risen to third summit, remaining air in sections 4 and 5 must escape by valves 3 and 4.

Assume that pressure on air valves will not exceed 10 lbs. gage. Then

$$\frac{\text{Gage pressure}}{\text{Atmospheric pressure}} = \frac{24.7}{14.7} = 1.68 \text{ and } V = 1049^*$$

For 1-in. air valves, $A = 0.7854$ sq. in. $= 0.00545$ sq. ft.
For $1\frac{1}{2}$-in air valves, $A = 1.767$ sq. in. $= 0.01227$ sq. ft.

Since passage through air valve is not an "orifice in thin plate," but more nearly a short pipe, take $C_o = 0.8$. (C_o = discharge coefficient)
Q for 1-in. air valve $= 0.8 \times 0.00545 \times 1049 = 4.58$ cu. ft. per sec.
Q for $1\frac{1}{2}$-in. air valve $= 0.8 \times 0.01227 \times 1049 = 10.30$ cu. ft. per sec.
100 ft. of 48-in. pipe has a content of 1257 cu. ft.

Since first depression is near first summit, the grade between is steep, and the cross-section of the pipe at this lowest point will soon be filled. As indicated in Fig. 329, suppose content of this depression equal to that of 125 ft. of pipe, or $1.25 \times 1257 = 1571$ cu. ft. To fill it will require $1571 \div 18 = 87$ sec. $= 1.5$ min. if water be admitted at rate of 18 cu. ft. per sec., found necessary by computations below to discharge air at above rates. If pressure of 10 lbs. be maintained at air valves 1 and 2, air will flow from them at $4.58 + 10.30 = 14.88$ cu. ft. per sec. Sections 2 and 3 as far as second summit contain together $36 \times 1257 = 45{,}252$ cu. ft. Neglecting slight difference in elevation between first and second summits, it will take 3037 sec. $= 50.6$ min. to fill the main to the second summit. For approximate computation third depression may be considered equivalent to first. This air will escape from valves 2 and 3 in $1571 \div 14.9 = 105.5$ sec. $= 1.8$ min. Remaining air in section 4 below third summit, say in $650 - 50 = 600$ ft. is 7542 cu. ft., and will pass from air valve 3 in 1640 sec. $= 27.3$ min. Assuming fourth depression, also equivalent to first, it will take $1571 \div 9.2 = 171$ sec. $= 2.9$ min. to get this air out through valves 3 and 4. This leaves about $1200 + 900 - 125 = 1975$ ft. of main, containing 24,826 cu. ft. of air to be discharged through two 1-in. air valves, which will require 2698 sec. $= 45$ min.

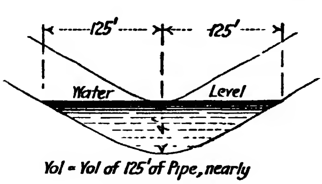


FIG. 329.

There is left part of section 3 between second summit and third depression, which is above the elevation of the third summit, about 400 ft., containing 5028 cu. ft. and requiring 488 sec. $= 8.1$ min. to fill. Section 1 has been filling meanwhile, but, probably, 900 ft. remain empty. Water may be admitted here more rapidly, say at 30 cu. ft. per sec., provided air has quite free egress at standpipe, as would be the case if standpipe and main were being filled together, or if there were an ample air valve at this end of the main. $9 \times 1257 \div 30 = 380$ sec. $= 6.3$ min. will complete the filling of the section.

* Air flow formula, p. 587.

Total time for filling main from standpipe to gate valve will be, therefore, the sum of these partial times, 143.5 min., or about 2 hrs. and 24 min.

To maintain conditions named above while filling the main, water should be admitted at the standpipe as follows: First 54 min. at 18 cu. ft. per sec.; next 27 min. at 10 cu. ft. per sec.; next 48 min. at 9¼ cu. ft. per sec.; next 8 min. at 10½ cu. ft. per sec. (or say 10 cu. ft. per sec. for 80 min.); last 6½ min. at 30 cu. ft. per sec. If rates of admission of water be materially increased for any period, pressure on air valves would probably exceed 10 lbs. per sq. in., and if valves were so constructed as to close when internal pressure exceeded this amount, there would be immediate danger of serious water ram. Hence, measure the water or place a pressure gage near or beyond first valve.

Loss of Head in Distribution System. The following coefficients were selected by the engineers of the Catskill aqueduct for pipes 36 to 72 in. in diam., in such condition as would result from being in service a number of years, and were intended to be conservative. Risers are smooth, concrete-lined vertical pipes (48 and 72 in. in diam.) connecting the high-pressure delivery tunnel in New York City with the subsurface main pipes.

Table 189. Losses of Head in Pipe Specials* in Terms of Velocity Head, $\frac{V^2}{2g}$

Special	V is velocity through	Factor
Central flow valve in riser...	Valve	0 5
Cross at top of riser, "Shaft Cap"..........	Riser	1 5
Gate valves, wide open, on mains of like diam.	Gate valve	0 1
Gates valves, wide open, with Venturi-shaped or standard increaser-shaped approaches.	Gate valve	0 3
Venturi meter	Throat of meter	0 14
90° bends (radius 2.5 to 3 times diam. of pipe)	Bend	0 4
Bends other than 90°, radius 2.5 to 3 times the diam. of the pipe, in direct ratio to loss in a 90° bend.		
Branches or other openings..............	Pipe passing opening.	0.03
Increaser	Small end. V_e = velocity in large end	$0.25\left(\frac{V^2}{2g} - \frac{V_e^2}{2g}\right)$

Average value of C_H for straight, cast-iron pipe in 12-ft. lengths, 100; for small velocities (up to 3 ft. per sec.), the corresponding $C = 93$; $C = 105$ for the highest velocities (over 10 ft. per sec.). For riveted, straight, steel pipe, $C_H = 80$ for velocities less than 2.5 ft. per sec., and 85 for higher velocities. The corresponding values of C are 78 and 92, respectively. Lock-bar straight steel pipe is taken the same as cast-iron. For the small risers, $C_H = 120$; in the larger terminal riser, loss is negligible up to a flow of 150 mgd.

Table 190. Velocities in Water Distribution Pipes

These velocities should not ordinarily be exceeded

Diam., in...	4	6	8	12	16	20	24	30	36	48
Velocity, ft. per sec. .	3 0	3 0	3 5	4 0	4 5	5 0	5 5	6 0	7 0	7.0

* Selected for use in designing Catskill aqueduct.

Economic Size of Force Main or Water Power Supply Pipe. (1) That pipe fulfills requirements of greatest economy wherein value of energy annually lost in frictional resistance equals 0.4 annual cost of pipe line.—(Arthur L. Adams.) Based on study of riveted steel pipes 30 in. to 66 in. diam., assuming

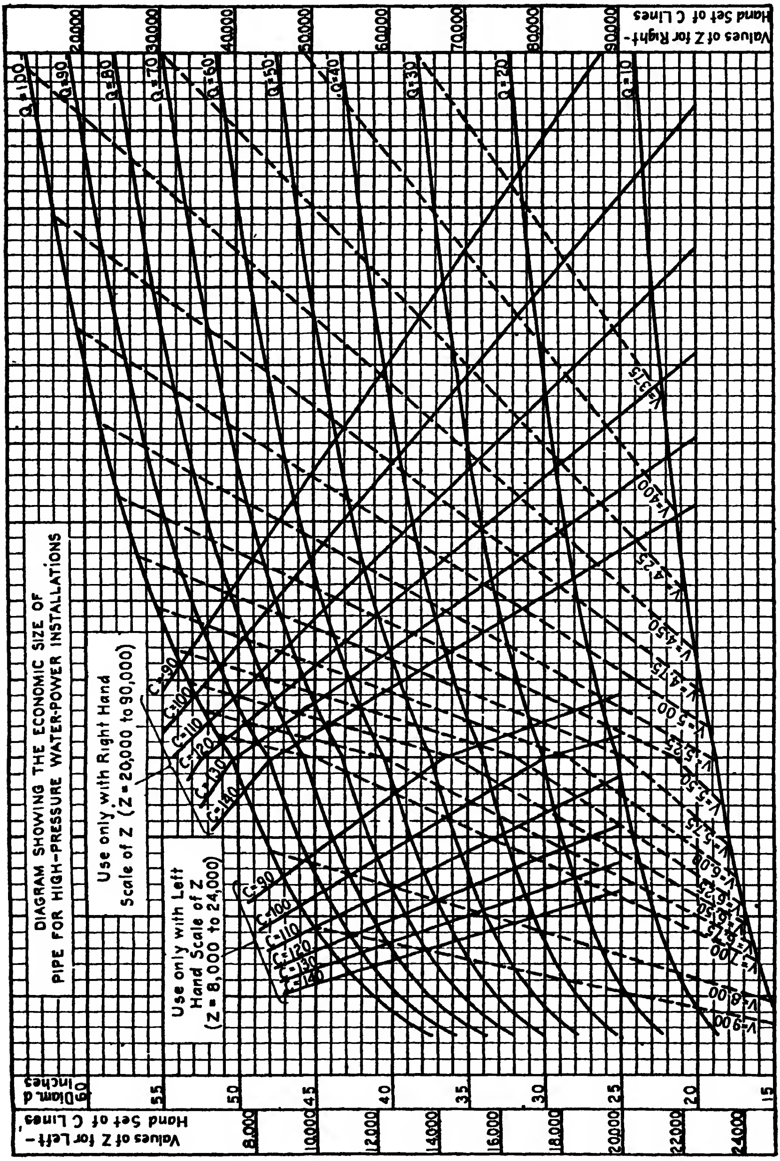


FIG. 330.—(Trans. Am. Soc. C. E., Vol. 59, 1907.)

that cost of pipe line + value of energy lost must be a minimum, that Chezy formula may be used with constant coefficient, that weights of pipes for given pressures are proportional to squares of diam.,

$$D = \sqrt[7]{\frac{0.0492Q^3u}{Hk - H_fk}} \quad \text{or } HkD^2 - H_fkD^2 = \frac{0.0492\,Q^3u}{D^{5*}}$$

or 2 × cost of pipe line = 5 × value of energy lost.

H = head on section under consideration, feet.
k = cost of pipe in dollars per lb.
u = value of energy of 1 c.f.s. with head of 1 ft.

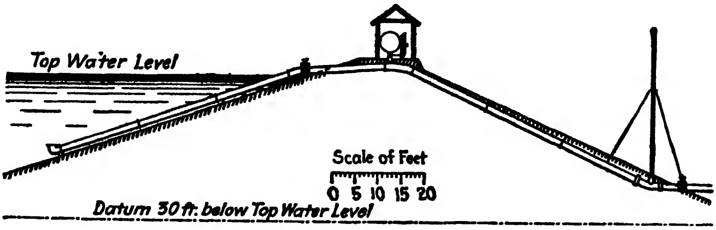


FIG. 331.
(T. A. S. C. E., Vol. 59, 1907, p. 63.)

H_f = total friction loss in section under consideration.
D = diam. in ft. Q = discharge, c.f.s.—(W. L. Butcher, T. A. S. C. E., Vol. 59, 1907.)

Buck's formula $d = 6.437\sqrt[7]{\frac{ZQ^3}{C^2}}$, $Z = \frac{U}{\text{cost per ft. of pipe}\dagger \div d^2}$

V = velocity, ft. per sec.
d = diam., inches
C = Chezy's C, see page 265.
U = the value of 1 hp.

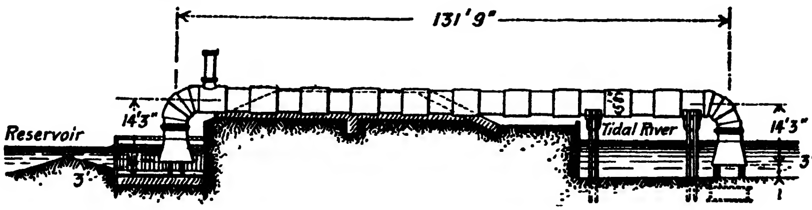


FIG. 332.
(T. A. S. C. E., Vol. 59, 1907, p. 65)

Enter Fig. 330 at value of Z (computed from assumed trial value of d), proceed horizontally to line for proper C, then vertically to Q line, and value of d is shown by horizontal lines. Recompute Z from this value of d, and repeat, if necessary. For Z from 8000 to 24,000, C lines at left must be used; for Z 20,000 to 90,000, those at right. Diagram may be used for compound pipes. Formula and diagram are both based on total theoretic power lost; actual value of hp. at wheels or pump × percentage of efficiency should be used. Break in curves at fourth heavy vertical is caused by change of horizontal scale.—(W. E. Buck, T. A. S. C. E., Vol. 59, 1907.)

* Values of D^5 on p. 554.
† See p. 390 to convert cost per lb. into cost per ft. Add laying and other costs.

Pipe Siphons. (True Siphons.) For drawing water from reservoir, over bank, etc. At E. London, Cape Colony, a 12-in. pipe gave following results on test for liberation of air. (See Fig. 331.) Capacity of air vessel, 650 gals.; difference of elevation between water in reservoir and air vessel, maximum 15 ft., minimum 6 ft., mean, 10.5 ft.; water discharged Feb. 12 to Mar. 22, 1906, 14 mg.; air liberated reduced to atmospheric pressure, 535 gals. = 0.0038 per cent., under mean vacuum of 10.5 ft.; mean temperature 70° F. (atmosphere). Water was drawn from an impounding reservoir and contained about 4 per cent. dissolved gases. Water was discharged at rate of 0.36 mgd. = 0.56 c.f.s.—(Chas. Anthony, Jr.)*

A 68-in. riveted steel pipe had, see Fig. 332, an air receiver at delivery end, from which a small steam jet extracted air; but a large quantity of air liberated was entrained and carried out with the discharged water. With 6-in. difference of water levels, delivery was very approximately 43 mgd. = 67 c.f.s.—(P. M. Pritchard.)*

Measurement by Elbows of Flow in Pipe Lines. $V = 5.60\sqrt[1.9]{H\frac{R}{D}}$. V = velocity, ft. per sec., R = mean radius of bend, ft., D = internal diam. of elbow, ft., and H = difference in hight of outer and inner water columns, feet; H is determined by taps at the inner and outer edges of the elbow with water columns. The elbows used in the experiments were ordinary 4- and 6-in. cast-iron elbows with bell joints, and $3\frac{1}{2}$-in. flanged cast-iron elbow; all had comparatively smooth interiors, and slight variations in cross-section. There is a consistent relation between the flow of water in an elbow and the difference in pressures between the convex and concave sides at the middle of the bend. The formula holds to within 2 per cent. for small elbows and probably holds for all sizes with slight modifications. The disturbed water returns to normal flow after a distance of 10 diam. The position of the taps should be radial and opposite at the middle of the bend and in a horizontal plane, but small departures from these conditions affect the results but slightly. If the elbow is in other than horizontal position, the gage reading must be corrected for the static head corresponding to the vertical distance between the taps. The position of the elbow with reference to gates and tees, if the latter are more than 10 diam. ahead of the elbow, is of no consequence.—(G. S. Jacobs and F. A. Sooy, Jl. of Elec., Power and Gas, Jul. 22, 1911.)

FLOW OF AIR

Formula for Inflow or Discharge.

$$Q = C_o A V$$

$$V = k(t_a + 273)\sqrt{\frac{n}{n-1}\left(\frac{1}{t+273}\right)\left(\frac{p}{a}\right)^{\frac{n-1}{n}}\left\{\left(\frac{p}{a}\right)^{\frac{n-1}{n}} - 1\right\}}$$

Q = discharge, cu. ft. per sec.
C_o = coefficient of flow
A = area, sq. ft.

* Trans. Am. Soc. C E, Vol. 59, 1907.

V = velocity, ft. per sec.
p = absolute pressure of internal air
a = absolute pressure of external air
t = temperature of internal air, degrees Centigrade
t_a = temperature of external air, degrees Centigrade
k = a constant = 76.344 for dry air
n = 1.41 for dry air; therefore $\frac{n}{n-1} = 3.451$; and $\frac{n-1}{n} = 0.291$.

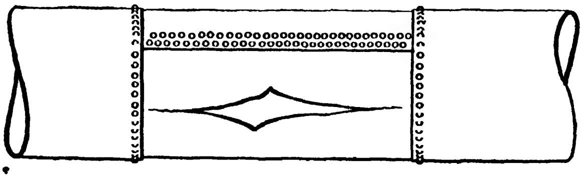


FIG. 333.

For $t = t_a = 17°$ C., or about $62\frac{1}{2}°$ F., the formula for V (for dry air) becomes:

$$V = 2415 \sqrt{\left(\frac{p}{a}\right)^{0.29} \left\{ \left(\frac{p}{a}\right)^{0.29} - 1 \right\}}$$

For $p = 14.7$ and $a = 12.7$, $V = 498.2$ ft. For $a = 10.7$, $V = 759$. For $a = 7.35$, $V = 1222$.*

To get diam. of inlets of air valves in pipe lines, C_o is assumed at 0.8, as these openings are not in thin plate, but "short pipes" fed by air valves. Hence, since $d^2 = \frac{4Q}{\pi C_o V}$, for $\frac{p}{a} = 2$, $V = 1222$, $d = 0.4336\sqrt{Q}$. (See p. 430 for Air Valves.)

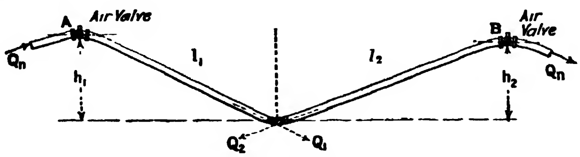


FIG. 334.

Air Inflow into a Pipe Line. Because of the thinness of some large diam. steel conduits, computations for the requisite air inflow to prevent collapse are made. *First assumption:* (*a*) The largest break might be tear shaped, as in Fig. 333, equal to $\frac{1}{4}$ the area of cross-section of the pipe. (*b*) With such a tear, the excess pressure under which the air should be fed should not exceed 2 lbs.; the vacuum within the pipe would be, therefore, only about $\frac{1}{7}$ full vacuum. *Second assumption:* The worst break might be occasioned by the washing out of the earth support from the pipe, and the tearing out of a section, thus allowing each branch to discharge freely. With such an extreme case, the pipe would withstand $\frac{1}{2}$ vacuum.

Under this second assumption (break of full area of section) the computation of the water discharged from either side in a depression and, therefore,

* Hütte, Taschenbuch.

of the air which must be admitted at the summit adjacent is simple: For $Q_1 = \beta\sqrt{s_1}$, and $Q_2 = \beta\sqrt{s_2}$, where $s_1 = \frac{h_1}{l_1}$, $s_2 = \frac{h_2}{l_2}$, and β is a constant from the formula for pipe flow. But Q_{air} at $A = Q_1 - Q_n$, since the normal flow of the conduit continues to feed over the summit A after the break and Q_{air} at $B = Q_2 + Q_n$ for the same reason. (Fig. 334)

If the break is not the full area of the pipe, the computations of air inflow are more difficult. For instance, in Fig. 335, where there is a break between two summits touching the hydraulic gradient:

$$S_1 = \frac{f_1}{l_1};\ S_2 = \frac{f_2}{l_2};\ Q_1 = \beta\sqrt{S_1};\ Q_2 = \beta\sqrt{S_2};\ Q_x = y\sqrt{h_x} = mA\sqrt{2g}\sqrt{h_x}$$

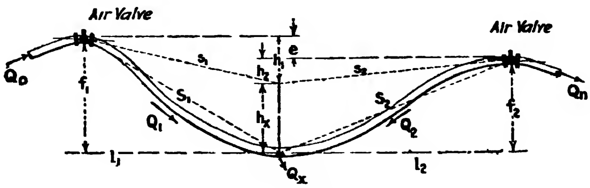


FIG. 335.

in which A = area of the break, and m = coefficient of discharge.

$Q_1 = \beta\sqrt{S_1}$, both when $Q_o = 0$ and when $Q_o = Q_n \gtreqless Q_x$.

$Q_2 = \beta\sqrt{S_2}$ when $Q_o < Q_x$; $Q_2 = 0$ when $Q_o = Q_x$ and is a minus quantity when $Q_o > Q_x$.

But $Q_1 + Q_2 = Q_x$, hence $\beta\left(\sqrt{\frac{f_1 - h_x}{l_1}} + \sqrt{\frac{f_2 - h_x}{l_2}}\right) = y\ \sqrt{h_x}$ (general).

The following is an approximation:

Let $\frac{f_1}{l_1} = S_1$; $\frac{f_2}{l_2} = S_2$; $l_1 + l_2 = l_0$.

$$Q_1 = \beta\sqrt{S_1 - \frac{2\,(S_1 + S_2)}{\left(\frac{y}{\beta}\right)^2 l_1 + \frac{l_0}{l_1^2 l_2} + \frac{4}{l_0 l_1}}}$$

$$Q_2 = \beta\sqrt{S_2 - \frac{2(S_1 + S_2)}{\left(\frac{y}{\beta}\right)^2 l_2 + \frac{l_0}{l_1 l_2^2} + \frac{4}{l_0 l_2}}}$$

Following is rigid solution: $\left(c = \left(\frac{\beta}{y}\right)^2\right)$

$$s_2 = \frac{\frac{1}{c}(f_1 - me)}{1 + 2\sqrt{\left(\frac{e}{h_2} + 1\right)\frac{l_2}{l_1} + \frac{l_2}{l_1}\left(\frac{c + l_1}{c}\right)}} = \frac{h_2}{l_2}$$ For e, see Fig. 335.

also

$$h_2 = \frac{l_2}{c}\left(\frac{f_1 - me}{1 + 2\sqrt{\left(\frac{e}{h_2} + 1\right)\frac{l_2}{l_1} + \frac{l_2}{c}m}}\right)$$

When h_2 from above is minus, the equations should be written with the other sign, because in this case $Q_x = Q_1 - Q_2$, and final equation becomes

$$h_2 = \frac{l_2}{c}\left(\frac{f_1 - me}{1 + 2\sqrt{\left(\frac{e}{h_2} - 1\right)\frac{l_2}{l_1} - \frac{l_2}{c}m}}\right)$$

Pneumatic Water Tanks. To find absolute pressure corresponding to a given volume (knowing original volume, and original absolute pressure) the temperature being the same, multiply the original absolute pressure by the original volume and divide by the given volume. This gives the final absolute pressure. If gage pressures are used or wanted, add 15 to the original gage pressure before multiplying, and from the final absolute pressure subtract 15. For example, suppose a pneumatic pressure tank when empty shows a gage reading of zero and we fill it two-thirds full of water compressing the air in it; if no air escapes, how much should the gage read? Call the volume of air at the beginning 100 per cent.; when the tank is two-thirds full of water, the air will be compressed to 33 per cent. of its original volume. Then the original absolute pressure will be 15. $15 \times \frac{100}{33} = 45$ lbs. absolute pressure. Subtracting 15 gives a gage pressure of 30 lbs. Suppose a tank is charged with

Table 191. Capacities and Pressures of Hydro-pneumatic Tanks Placed with Axis of Cylinder Horizontal*

Example. To find gallons of water in tank 5 ft. diam 20 ft. long, with water level 36 in. above bottom of shell, column one, opposite 36 under 60 in. find 92 gals., multiplied by 20 equals 1840 gals; pressure equals 52.2 lbs. per sq. in.

Hight of water, in. (A)	Diameter of tank (B), capacities in gals. per lin ft. for each inch in hight (A), and pounds pressure by inches in hight when initial pressure equals 10 lbs per sq. in.											
	48 in.		60 in.		72 in.		84 in.		96 in		108 in.	
	Gals	Press.	Gals.	Press.	Gals	Press	Gals	Press.	Gals.	Press.	Gals.	Press
12	18	16 0	21	14 2								
13	20	16 9	23	14 8								
14	23	18 0	26	15 4	29	13 9						
15	25	19 1	29	16 1	32	14 4						
16	27	20 3	31	16 8	35	14 9	38	13 7				
17	30	21.5	34	17 6	38	15 5	41	14 1				
18	32	23 0	37	18 6	41	16 0	45	14 6	49	13 7		
19	35	24 5	40	19 5	45	16 6	49	15 0	53	14 0		
20	37	26 3	43	20.4	48	17.2	52	15 5	57	14 4	61	13 6
21	40	28 2	46	21 4	51	17 9	56	16 0	61	14 7	65	13 9
22	42	30.2	49	22 5	55	18 6	60	16.5	65	15.2	70	14.3
23	45	32.7	52	23 6	58	19.4	64	17 1	69	15 5	74	14 6
24	47	35 0	55	24.9	62	20 2	68	17.6	73	16.0	79	15.0
25	49	37.7	58	26 3	65	21 1	72	18 2	78	16 5	83	15 3
26	52	40 9	61	27.7	68	21 9	76	18 8	82	16 9	88	15 7
27	54	44 5	64	29 4	73	22 9	80	19.5	87	17.4	93	16.1
28	57	48 3	67	31.0	76	23 9	84	20 2	91	17 9	98	16 5
29	59	53 0	70	32 6	80	24 9	88	20 9	95	18 5	103	16 9
30	62	58 0	73	35 0	83	26 2	92	21.7	100	19 0	108	17 3
31	64†	63.8	77	37.4	87	27 3	96	22.5	105	19.6	112	17.7
32	66	70.5	80	39.7	91	28.7	101	23.4	109	20.2	118	18.2
33	69	78.2	83	42.4	95	30 0	105	24.2	114	20 9	123	18 6
34	71	88 0	86	45.4	98	31 5	109	25 1	119	21 6	128	19.3
35	73	99 0	89	48.5	102	33.0	113	26 2	124	22 1	133	19 8
36	. . .	.	92	52.2	106	35.0	117	27.1	129	22 9	138	20.3
37	. . .	. .	95	56.0	110	36.9	122	28 3	134	23 6	144	20 6

*From catalog of Ralph B. Carter Co., N. Y. City. † At this hight tank is two-thirds full.
Continued on next page.

Table 191. Capacities and Pressures of Hydro-pneumatic Tanks Placed Horizontal.—(*Continued*)

Hight of water, in (*A*)	Diam. of tank (*B*), capacities in gals. per lin. ft. for each inch in hight (*A*), and pounds pressure by inches in hight when the initial pressure equals 10 lbs. per sq. in.											
	48 in.		60 in.		72 in.		84 in.		96 in.		108 in.	
	Gals	Press	Gals.	Press.	Gals.	Press.	Gals.	Press.	Gals.	Press.	Gals.	Press
38			98*	60.2	113	38.7	126	29.4	138	24.4	150	21.5
39			101	65.0	117	40.9	131	30.7	144	25.3	155	22.1
40			104	70.5	121	43.2	134	31.8	149	26.3	160	22.6
41			106	76.8	125	45.7	139	33.4	153	27.1	166	23.5
42			110	84.0	128	48.2	144	35.0	158	28.0	171	24.1
43			113	92.5	132	51.3	148	36.5	163	28.9	176	24.7
44			..	...	136	54.3	153	38.3	168	30.2	182	25.5
45			.	....	139	57.7	157	39.9	173	31.2	188	26.3
46					142*	61.8	162	41.8	178	32.3	193	27.1
47					146	66.0	166	43.9	183	33.5	198	27.9
48					150	70.5	171	45.3	188	35.0	204	28.9
49					154	76.0	175	48.4	194	36.3	209	30.0
50					157	81.8	179	51.0	199	37.9	215	30.8
51					161	86.5	183	53.5	203	39.6	220	31.7
52					164	95.7	187	56.2	208	41.1	225	33.2
53							192*	60.0	213	42.5	232	33.9
54							196	63.0	218	44.4	238	35.0
55							200	66.6	223	46.3	245	36.2
56							204	70.5	228	48.2	250	37.3
57							208	75.2	232	50.3	255	39.1
58							212	79.8	238	52.8	261	40.4
59							217	84.5	243	55.2	267	42.1
60							221	91.5	247	57.8	273	43.3
61									252*	60.0	279	45.0
62									257	64.0	285	47.5
63									262	66.3	290	48.3
64									266	70.8	295	50.5
65									270	74.7	300	52.5
66									275	79.0	306	54.5
67									280	83.5	312	57.0
68									285	88.0	318*	60.0
Full	94		147		212		288		376		476	

* At this hight tank is two-thirds full

compressed air so that with no water in it, the gage reads 20 lbs. Then pump the tank two-thirds full of water; what will the pressure be? If we call the original volume of air 100 per cent. the final volume is 33 per cent. The original absolute pressure is $20 + 15 = 35$ lbs.; $35 \times \frac{100}{33} = 105$ lbs. absolute pressure. Subtracting 15 gives a final gage pressure of 90 lbs.

Air in Water. Rate of rise of air bubbles through water. In flowing through gate-houses and other structures, under certain circumstances, water entrains air, and it is sometimes necessary to remove excess air before permitting water to flow on into an aqueduct or pipe. In connection with design of an air remover for Catskill aqueduct (see E. R., Aug. 3, 1912) to determine rate at which air would probably separate from water, simple experiments were made (1909), with results below, on rate at which air bubbles would rise through water. A wooden tank 6 in. square inside, 11 ft. 4 in. high, with panes of glass at intervals on one side, and a small pipe near bottom for admitting air under pressures up to 5 lbs. was used. The air pipe was horizontal with end

capped, one hole $\frac{1}{16}$ in. diam. in top of pipe at vertical center line of tank; air was controlled by a cock. Water temperature was 70° F., air 72°.

Table 192. Rate of Rise of Air Bubbles through Water

No. of expts. in av.	Estimated size of bubble, in	Av. time, minutes, consumed in rising: 5 ft	Av. time, minutes, consumed in rising: 10 ft	Remarks
6	$\frac{1}{64}$	0 24	0 47	Bubbles liberating themselves naturally from orifice in top of air pipe.
2	$\frac{1}{32}$	0 17	0 30	
4	$\frac{1}{16}$	0 15	0 31	
3	$\frac{3}{32}$	0 14	0 27	
4	$\frac{1}{8}$	0 12	0 24	
3	3	0 05	0 11	Large hemispherical bubbles, made by releasing large quantity of air quickly, rising at head of large groups of smaller bubbles.
6	4	0 04	0 10	
1	5	0 05	0 10	
7	$\frac{1}{64}$	0 24	0 47	Time is that required, after liberating large quantity of air, to clear water of all bubbles larger than size stated.
4	$\frac{1}{32}$	0 20	0 41	
2	$\frac{1}{16}$	0 17	0 34	
5	$\frac{1}{8}$	0 13	0 24	
2	$\frac{1}{4}$	0 11	0 22	

Water discharged from a hose immersed about 5 ft. in water liberated air in bubbles of various sizes, including a quantity of very small ones $\frac{1}{64}$ in. and less. No attempt was made to measure rate of rise, but it was noted that the smaller bubbles rose very slowly. Water was discharged from $\frac{1}{4}$-in. nozzle on to the water surface in the tank. It carried air with it to a depth of 6 to 8 in. in form of bubbles from $\frac{1}{8}$-in. diam. to very small ones. There seemed to be a distinct division of bubbles into two groups, those about $\frac{1}{16}$ in. in diam. and over, and those about $\frac{1}{64}$ in. and under. There were very few bubbles of diam. between $\frac{1}{16}$ and $\frac{1}{64}$ in. (Pressure at nozzle probably about 15 lbs.)

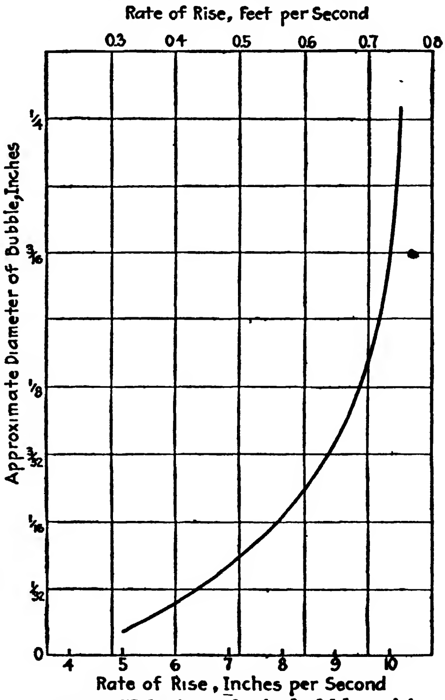


FIG. 336.—Velocity of air bubbles rising through water.
(Experiments at B. W. S. Laboratory, Aug., 1909.)

The velocity of "commotion" bubbles (about $\frac{1}{4}$ in. diam.) is about 9 ft. per sec. The velocity varies as the square root of the diam., and the diam. increases as the bubble approaches the surface due to the lessening pressure. "Elimination" bubbles are seldom over 0.01 in. diam. at any stage of their ascent, and cannot rise at a rate greater than 2 in. per sec. (Frizell's "Water Power," p. 523.)

Experiments made with glass tubes 5 in. in diam. showed a velocity of

8 to 10 in. per sec. for ⅜-in. bubbles. The velocity does not depend on the depth of water.

WATER-HAMMER IN PIPES

Stresses in Pipe. When water flows through a very long pipe, sudden closing of valves, especially the last quarter of travel, tends to cause high momentary bursting pressures, or water-hammer; extreme pressure would be occasioned by instantaneous closing. If the pipe does not move lengthwise, kinetic energy of water will exhaust itself in compressing water and distending walls of pipe. Distention of pipe may be neglected. Water has only one modulus of elasticity, "Bulk Modulus," or E_b. If water, of original volume B, is by compression from all sides reduced by dB, fluid pressure induced being P_r lbs. per sq. in., then:

$$E_b = \frac{P_r}{dB \div B} = \frac{P_r B}{dB}$$

For pressures below 1000 lbs. per sq. in. (and at ordinary temperatures) E_b may be taken as 294,000 lbs. per sq. in. (For very high pressure see E. N., Oct. 4, 1900.)

$$P_r = V\sqrt{\frac{E_t w}{g}}$$

w = unit weight of water, and V = initial velocity. "Excess pressure" is proportional to original velocity V of water in pipe.

Incidentally, velocity of sound in water:

$$U = \sqrt{\frac{E_b g}{w}}$$

With $E_b = 294{,}000$ lbs. per sq. in., $g = 32.2$ and $w = 62.5$ lbs. per cu. ft. $U = 4670$ ft. per sec. and P_r (in lbs. per sq. in.) $= 63 \times V$ (in ft. per sec.)

Under unit pressure P_r at this instant acting also as bursting pressure radially outward on inner surface of pipe wall, simultaneous tensile stress (or "hoop-tension") in pipe wall, p_n lbs. per sq. in., will have a value of $p_n = rp_r \div t$, where t = thickness of pipe wall, and r = internal radius of pipe. If E is modulus of elasticity (linear, Young's modulus) of metal of pipe, and L_p = increase of length of circumference due to stress p_n, and r_s = corresponding increase in radius,

$$E = p_n \div (L_p \div 2\pi r), \text{ or } E = (rP_r 2\pi r) \div L_p t)$$

$$U = \sqrt{\frac{g}{w} \cdot \frac{EE_b t}{(tE + 2rE_b)}}$$

(diminished) velocity of sound along water in pipe, distention of latter being considered; therefore

$$P_r = V\left(\sqrt{\frac{wEE_b t}{g(tE + 2rE_b}}\right)$$

is "excess pressure" tending to burst pipe.—("Hydraulic Motors," I. P. Church, 1908.)

Surges in Pipe Lines. The increased stress in lbs. per sq. in. of metal, S_1, due to the sudden stoppage of water $= 307\ V\sqrt{\frac{d}{t}}$, where $V =$ velocity of flow in ft. per sec. in the pressure pipe just before the interruption; $t =$ maximum thickness of pipe, inches; $d =$ nominal diam. of pipe, inches. Example: In a 60-in. pipe, 0.5 in. thick at the lower end, what is the fiber stress due to the sudden stopping of water flowing at a velocity of 5 ft. per sec.? $\frac{d}{t} = 120$, and $S_1 = 17{,}000$ lbs. per sq. in. The sum of the initial stress, S, due to the static head, and S_1, gives the required stress.

Excess pressure, in ft. head $= 1415 V \sqrt{\frac{t}{d}}$. This excess pressure is independent of the length of pipe, because each foot of pipe which adds to the energy of the water also adds to the length of pipe on which work is done. These equations are for instant closure of gates, and show the necessity for slow closing. If the energy imparted to the pipe is neglected, an assumption on the side of safety, the rise in pressure due to the gradual closing of the valves $= 0.0135\ La$. In this case the rise in pressure is directly proportional to the length of pipe, L, and to the rate of bringing the water to rest. In other words, a the rate of acceleration, $= \frac{\text{velocity}}{\text{time}}$.—(F. G. Baum, E. R., Dec. 24, 1910, p. 739.)

Closing of Valves. Water-hammer is the more severe the quicker the closing, the higher the velocity of the water and the greater the length of the moving column of water. Sudden closing of an automatic balanced valve in a short 42-in. steel pipe, at Springfield, Mass. filters, static head 25 ft., caused hammer estimated at 10 times this head, forcing water through riveted joints in a connected ¼-in. steel plate water-wheel case. (A. Hazen, J. N. E. W. W. A., 1911.) At 2.53 miles from reservoir, Rochester, N. Y., E. Kuichling repeatedly observed increased pressures in 24-in. cast-iron pipe caused by hand closing in 20 min. of a 24-in. valve, 2100 ft. beyond the point of observation; hydraulic pressure, valve open, 40.2 lbs.; velocity 3.45 ft. per sec.; static pressure, valve shut, 49 lbs. Last 4 min. of closing, pressure rose to maximum of 65 lbs., then oscillated rapidly a few seconds 40 to 65 lbs., then gradually reduced in 2 min. to 42 to 57 lbs., and in 8 min. more to 46 to 52, with increasing intervals between pulsations; at end of 15 min. pressure varied little from 49 lbs. Concluded that similar closing of valves in long and large pipes would not induce internal pressures greater than 1.5 times static pressure. This is a convenient way of making allowance, but there is no direct relationship between static pressure and water-hammer. Hydraulic elevators, locomotive standpipes and similar devices are prolific causes of water-hammer. Air chambers, relief valves and open standpipes extending above normal hydraulic gradient are means for preventing damage. Slow closing of valves is best prevention.

Joukovsky's Experiments.* Joukovsky investigated as special cases of water-hammer (see 1898 memoires de L'Academie Imperiale des Science de

* O. Simin, Proc. Am. W. W. Assn., 1904.

St. Petersburg) (1) effect of dead ends in increasing shock; (2) effect of rate of closing valve; (3) air chambers; (4) safety valves; (5) influence of air pockets and of leaks upon form of pressure curve. Tests were made on 2-in., 4-in., 6-in., and 24-in. cast-iron pipes, 2494, 1050, 1066 and 7007 ft. long, respectively. Formula for velocity of propagation of pressure wave, for any system of units:

$$V_\omega = \frac{1}{\sqrt{\frac{D}{E_b} + \frac{d}{t}\frac{D}{E}}}$$

V_ω = velocity of propagation of wave, ft. per sec., V = extinguished velocity of flow, d = diam. of pipe, inches. t = thickness of pipe, inches. D = density of water = unit weight ÷ acceleration of gravity, E = Young's linear modulus of elasticity for material in pipe, 14,000,000 for cast-iron, E_b = volumnar (bulk) modulus of elasticity of water, say 300,000 lbs. per sq. in.

Comparison of results of computations and tests

d	$V\omega$ computed	$V\omega$ from tests
2	4424	4375
4	4288	4200
6	4116	4100
24	2996	3313

Additional pressure, P_r, produced by closure of valve:

$$P_r = VDV_\omega$$

Increase of unit pressure stands in direct proportion to extinguished velocity, V; it is independent of length of pipe, and directly proportional to velocity, $V\omega$, of propagation of pressure wave. Substituting numerical values, Joukovsky computed the following values of additional shock pressure:

Diam. of pipe, in.	Additional pressure per ft.-per-sec. of extinguished vel., in atmospheres
2	4.066
4	3.886
6	3.783
24	2.754

Miss Simin summarizes Joukovsky's results: 1. Shock pressure is transmitted with constant velocity seemingly independent of intensity of shock, but dependent upon elasticity of material in pipe, and upon ratio of thickness of pipe to diam. This velocity is about 4200 ft. per sec. for 2-, 4-, and 6-in. diam., and 3290 ft. for 24-in. 2. Shock pressure is transmitted with constant intensity; is proportional to extinguished velocity of flow and to speed of propagation of pressure wave. For 2-, 4-, and 6-in. pipes, increase of pressure for every ft. per sec. of extinguished velocity of flow is about 4 at-

mospheres, and about 3 atmospheres for 24-in. 3. Phenomenon of periodical vibration of shock pressure is explained by reflection of pressure wave from ends of pipe. 4. If water column continues flowing, no noticeable influence upon shock pressure takes place; pressure wave is reflected from open end of pipe, in same way as from reservoir with constant pressure. 5. Dangerous increase of shock pressure occurs when pressure wave passes into pipe of smaller diam., containing a dead end; when wave is reflected from a dead end, shock is doubled. If several dead ends branch from a dead-ended trunk, pressures may become very great. 6. Simplest method of protecting from water-hammer is use of slow-closing valves; duration of closure to be proportioned to length of pipe line. Air chambers, if of ample size, placed near valves, tend to eliminate hydraulic shock; but it is difficult to maintain the air supply. Safety valves allow to pass through them only waves of intensity corresponding to elasticity of springs of valve. 7. Diagrams of shock pressures are of value in determining location and extent of air pockets, and location of leaks.

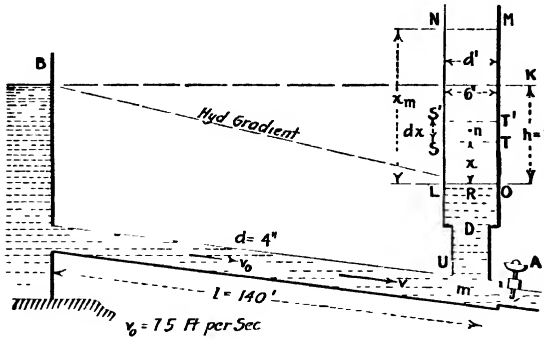


FIG. 337.

Surge in Hydraulic Standpipe.* In each experiment steady flow was first secured with *LO* 11 ft. below *BK*, corresponding hydraulic grade line being *BR*. Let $\frac{1}{4}\pi d^2$, sectional area of main pipe, be denoted by *F*, and that of standpipe (at *LO* and above), viz., $\frac{1}{4}\pi d'^2$, by *F'*. Let the valve at *A* be instantaneously closed. Surface of water in the standpipe immediately begins to rise above its initial position *LO* and velocity, v, of the water in the main pipe begins to diminish from initial value $v_0 = 7.5$ ft. per sec. Unsteady flow now sets in; what value of v will correspond to any value of x (hight of surface *ST* in standpipe, above *LO*), during upward "surge;" *i.e.*, find v as a function of x; and finally determine greatest hight $x_m = \overline{OM}$ of surge, the water in both pipes having then come to rest (for an instant). Let γ denote weight of a cubic foot of water, and g acceleration of gravity. Apply Principle of Work and (Kinetic) Energy, as governing the motion of an assemblage of rigid bodies, to the movement, in time increment, dt, of all particles of water in supply pond, main pipe, and standpipe; their aggregate weight being G lbs. Equating work of working force G to friction work plus change of kinetic

* Irving P. Church (Cornell Civil Engr., Dec., 1911). Experiments, 1909–1910, Hydraulic Lab., Cornell Univ., by H. H. Conway and F. H. Storey.

energy, all forces besides gravity and friction being either neutral or self-cancelling (atmospheric pressure)

$$\gamma F'dx(h - x) = \gamma \frac{Fl}{g} vdv + \gamma \frac{F'x}{2g}\left[1 + \frac{4fl}{d}\right]^* \tag{1}$$

Dividing through by $F'\gamma$, also denoting $\frac{1}{2g}\left[1 + \frac{4fl}{d}\right]$ by A, and $\frac{F}{F'} \cdot \frac{l}{g}$ by B,

$$[h - x - Av^2]dx = Bvdv \tag{2}$$

$$\text{Integrating, } Av^2 = h - x + \frac{B}{2A}\left[1 - \epsilon^{-\frac{2A}{B}x}\right] \tag{3}$$

which is the desired relation giving velocity v in the pipe for any value of x during the (first) upward motion, or "surge," of the surface ST of the water in the standpipe. Position reached by this surface at end of first surge is found by putting v = zero in (3) and solving for x, which is then called $x_m = \overline{OM}$ in Fig. 337.

$$x_m = h + \frac{B}{2A}\left[1 - \epsilon^{-\frac{2A}{B}x_m}\right] \tag{4}$$

This transcendental relation can be solved for x_m only by trial; not tedious, since the value of the bracket is not greatly affected by changes in x_m as successive assumptions are made. ϵ is 2.71828, Naperian base.

Applying eq. (4) to experiment with 6-in. standpipe (d' = 6 in.), we have (since h = 11 ft., = Av_0^2 and v_0 = 7.5 ft. per sec.), A = 0.1955; while

$$B = \frac{F}{F'} \cdot \frac{l}{g} = \frac{(4)^2}{(6)^2} \cdot \frac{140}{32.2} = 1.932;$$

both values involving foot and second as units.

Hence

$$\frac{B}{2A} = 4.94 \text{ ft.; while } \frac{2A}{B} = 0.2024.$$

With 20 ft. as first trial value for x_m,

$$\epsilon^{-\frac{2A}{B}x_m} \text{ becomes } \epsilon^{-0.2024 \times 20} = \epsilon^{-4.048}$$

and hence it is reciprocal of number whose natural logarithm ($\log_e$) is 4.048, which is 57.

From eq. (4)

$$11 + 4.94\left[1 - \frac{1}{57}\right] = 15.85 \text{ ft.,}$$

which is much smaller than assumed 20 ft.

Assuming 16 ft. for x_m, right-hand member of (4) = 15.7; assuming 15.7 ft. for x_m, from (4)

$$x_m = 11 + 4.94\left[1 - \frac{1}{24}\right] = 15.74 \text{ ft.}$$

Result of actual experiment was x_m = 16 ft.

*f = friction factor, see page 556.

Experiment with 4-in. standpipe, $h = 11$ ft. and $v_0 = 7.5$ ft. per sec.; A, as before $= 0.1955$; but since now diam. of standpipe is equal to that of main pipe, so that $F = F'$,

$$B = \frac{140}{32.2} = 4.348.$$

$$\frac{B}{2A} = 11.12 \text{ ft.; and } \frac{2A}{B} = 0.0900.$$

Assume $x_m = 25$ ft., right-hand member of eq. (4) becomes

$$11 + 11.12\left[1 - \frac{1}{9.4}\right] = 20.93 \text{ ft.}$$

Finally value 20.3 ft. proves sufficiently close. Value found by experiment was 19.8 ft.

For practical use, transform eq. (4) by writing $\frac{1}{K} = \frac{2A}{B}$ in exponent inside bracket, while restoring original meaning of A and B *outside* bracket,

$$x_m = h + \frac{F}{F'} \cdot \frac{l}{1 + \frac{4fl}{d}}\left[1 - e^{-\frac{x_m}{K}}\right] \tag{5}$$

(Note that the coefficient outside the bracket $= K$.)

In solving eq. (5) for x_m in any actual case of a water-power plant with a long supply pipe, a fair value for the first approximation may usually be obtained by neglecting second term in bracket, since value is generally small compared with unity. As to *time* occupied in first upward surge, observed time in case of 6-in. standpipe was 10 sec.; and for 4-in. standpipe, 5 sec. A relation holding between v and x for any instant during return motion (first *downward* surge) of water in standpipe (with corresponding "backward" motion of water in main pipe) can easily be established by use of foregoing methods, it being remembered that initial conditions of this downward surge are final conditions of first upward surge. Another upward surge now follows, and so on; but each succeeding surge is shorter in range than that immediately preceding until finally water in standpipe comes to rest with summit on level with that of lake surface. See also Trans. Am. Soc. C. E., Vol. 78, 1915, p. 760, R. D. Johnson.

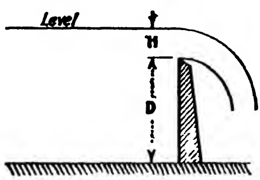


FIG. 338.

WEIRS

Bazin's Weir Formula. (Rectangular Weirs.) Bazin's results per linear foot of weir, for weirs with sharp crests,* without end contractions, are approximated by:

$$Q = c\left[1 + 0.55\left(\frac{H}{D+H}\right)^2\right]H^{\frac{3}{2}}\sqrt{2g}$$

$$c = 0.405 + \frac{0.003}{H}$$

*See page 602 for flat crests.

or more roughly

$$Q = c'H^{\frac{3}{2}}\sqrt{2g}$$

$$c' = 0.425 + 0.21\left(\frac{H}{D+H}\right)^2$$

D and H are in ft., Q in cfs. (Fig. 338).

Weir Box is used to measure the discharge of small pumps, and other relatively small quantities. The depth on the weir must not be over one-third the depth of the box or canal, and the distance of the side of the weir opening from the side of the canal or box must not be less than the depth on the weir. The edge over which water falls must be level. All edges of the notch must be sharp. Since a common method of measuring the head on weirs is to use an ordinary foot-rule divided into inches and sixteenths, and the discharge in gallons per minute is often wanted, the following table has been prepared by the American Well Works, based on Francis' formula, for end contractions:

$Q = 3.33\,[L - 0.2H]\,H^{\frac{3}{2}}$, for weirs from 1 to 20 ft. long.

Table 193. Rectangular Weir Discharges, End Contractions

Figures at left and top are depths or heads, figures in body of table, gals per min.

Head	0"	1/8"	1/4"	3/8"	1/2"	5/8"	3/4"	7/8"
12-in Weir with Full Contractions								
0"		1.58	4.48	8.33	12.67	17.65	23.14	29.09
1"	35.5	42.3	49.4	56.9	63.2	72.7	81.1	89.8
2"	98.7	107.8	117.2	126.8	136.7	146.8	157	167.5
3"	178	189	200	211	223.	234	246	257
4"	269	282.	294	306	319	331	344	357
5"	370	383.	396	409	423	436	450	464
6"	477	491.	505	519	533.	547.	562.	576
7"	590	605	619	634	649	663	678	693
8"	708	723	737	752.	767	783	798	813
9"	828.	843	859	874	889	905	920.	935
10"	951	966	982	997.	1013	1028	1044	1060
11"	1075	1091	1106	1122	1137	1153	1169	1184
12"	1200					..	.	
2-ft Weir with Full Contractions								
	0"	1/8"	1/4"	3/8"	1/2"	5/8"	3/4"	7/8"
0"	.	3.18	8.98	16.72	25.44	35.48	46.58	58.62
1"	71.5	85.4	99.8	114.0	127.9	147.4	164.6	182.5
2"	200.8	219.6	238.0	258.9	279.3	300.3	321.6	343.4
3"	366	388	411	435	459	483	508.	533
4"	558	584	610	636	663	690	718	745
5"	773	802	830	859	888	918	947	977
6"	1008	1038	1069	1100	1131.	1163.	1194	1226.
7"	1259	1291.	1324	1357.	1390	1423	1456	1490
8"	1524.	1558	1593	1627.	1662	1697	1732	1767.
9"	1802.	1839.	1874.	1910.	1946.	1982.	2019.	2055.
10"	2092.	2129.	2166.	2203.	2241.	2278.	2316.	2354.
11"	2392.	2430	2468.	2506	2543.	2584.	2622	2661.
12"	2700.							

Table 193. Rectangular Weir Discharges, End Contractions.—(*Continued*)

Figures at left and top are depths or heads; figures in body of table, gals. per min.

3-ft Weir with Full Contractions

	0"	⅛"	¼"	⅜"	½"	⅝"	¾"	⅞"
0"	. . .	5 0	13 5	25 0	38 0	53 5	70	88.
1"	107.5	128.5	150.5	172 0	192.5	222.	248.	275.
2"	303.	331.5	360.	391.	422.	454.	486.	519.
3"	554.	588.	622.	659.	695.	732.	770.	808.
4"	846.	886.	926.	966.	1008.	1049	1092.	1134
5"	1177.	1220.	1264.	1308.	1354.	1400.	1445.	1492
6"	1538.	1585.	1632.	1681.	1730.	1778.	1828.	1877.
7"	1927.	1978.	2028.	2080.	2131.	2182.	2235.	2288
8'	2341.	2394.	2448.	2502.	2556.	2611.	2666.	2721
9"	2776.	2835.	2889.	2946.	3002.	3060.	3117.	3175.
10"	3233.	3292.	3350.	3409.	3468.	3528.	3588.	3648.
11"	3708.	3769.	3830.	3890.	3949.	4014.	4076.	4138.
12"	4200.						......	

4-ft. Weir with Full Contractions

	0"	⅛"	¼"	⅜"	½"	⅝"	¾"	⅞"
0"	......	6 5	18	33 5	51 0	71 0	93 5	117 5
1"	143.5	171.5	201	230 5	257 5	297.0	331 5	368
2"	405.	443.	481.	523.	564.5	607.5	651.	695.
3"	741.	787.	834	882.	932.	981.	1032.	1083
4"	1136.	1188.	1242.	1297.	1352.	1408.	1466.	1522
5"	1580.	1639.	1698.	1758.	1819.	1880.	1942.	2005.
6"	2068.	2132.	2196.	2262.	2328.	2394.	2460.	2527.
7"	2595.	2664.	2733.	2802.	2872.	2942.	3014.	3086.
8"	3158.	3230.	3302.	3376.	3450.	3525	3600	3675.
9"	3751.	3830.	3904	3982	4060	4138.	4216.	4294.
10"	4374.	4454.	4534	4615	4696	4778	4859.	4942
11"	5025.	5108.	5191	5275	5355.	5444	5529	5614.
12"	5700	.		. . .			.. .	

Table 194. Rectangular Weir Discharges, without End Contractions

(Francis Formula, W. S. and Irr. Paper, No. 200)

Depth of water on weir, in.	Discharge per lin ft. of weir		Depth of water on weir, in.	Discharge per lin ft of weir		Depth of water on weir, in	Discharge per lin ft of weir		Depth of water on weir, in.	Discharge per lin ft. of weir	
	Cu ft per sec.	Gals per min		Cu ft per sec	Gals per min		Cu ft per sec	Gals per min		Cu ft per sec.	Gals. per min.
1	0 080	35 951	6	1 1773	528 409	11	2 9226	1311 8	16	5 1268	2301
1¼	0 112	50.269	6¼	1 2523	562 071	11¼	3 0228	1356 7	16¼	5 2477	2355
1½	0 147	66 068	6½	1 3275	595 823	11½	3.1240	1402 1	16½	5 3691	2410
1¾	0 185	83.213	6¾	1 4048	630 518	11¾	3.2267	1448 2	16¾	5 4916	2465
2	0 226	101 615	7	1 4836	665 886	12	3.3300	1494 6	17	5 6150	2520
2¼	0 270	121.319	7¼	1.5638	701 882	12¼	3 4345	1541.5	17¼	5 7393	2577
2½	0 3167	142.145	7½	1.6454	738 5	12½	3.5403	1589 0	17½	5 8645	2632
2¾	0.3654	164.003	7¾	1.7282	775 7	12¾	3 6470	1636.9	17¾	5 9908	2689
3	0.4162	186 803	8	1.8126	813.6	13	3.7556	1685 6	18	6 1176	2746
3¼	0.4693	210.636	8¼	1.8983	852.0	13¼	3.8638	1734 2	18¼	6.2453	2803
3½	0 5245	235.412	8½	1 9852	891.0	13½	3 9735	1783	18½	6.3738	2861
3¾	0 5817	261 085	8¾	2 0736	930 7	13¾	4.0842	1833	18¾	6.5039	2919
4	0 6408	287.611	9	2.1629	970 8	14	4.1963	1883	19	6.6344	2977
4¼	0.7020	315 079	9¼	2 2537	1011.5	14¼	4.3092	1934	19¼	6.7660	3037
4½	0.7647	343.221	9½	2.3456	1052.8	14½	4.4230	1985	19½	6 8980	3096
4¾	0.8292	372.171	9¾	2 4389	1094.7	14¾	4.5382	2037	19¾	7 0309	3155
5	0 8956	401.973	10	2 5332	1137.0	15	4.6538	2089	20	7 1642	3215
5¼	0 9636	432.494	10¼	2.6289	1180 0	15¼	4.7673	2140	20¼	7 2997	3276
5½	1.0333	463.777	10½	2 7256	1223.3	15½	4.8880	2194	20½	7.4354	3337
5¾	1 1046	495.779	10¾	2 8234	1267 2	15¾	5 0071	2247	20¾	7 5720	3398

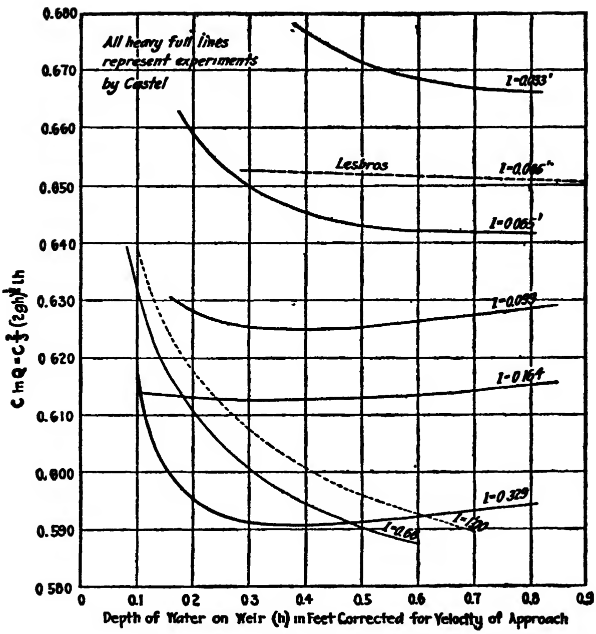


FIG. 339.—Coefficient C for short sharp-crested rectangular weirs, with end contractions.
(Plates VII and IX, "Hydraulics," Hamilton Smith, Jr.)

Triangular Weir. *Discharge:*

Angle at vertex 90°, $Q = 2.54\ h^{\frac{5}{2}}$ (Prof. Thomson gives 2.64)

Angle at vertex 90°, $Q_m = C_m h^{\frac{5}{2}}$, in which C_m has following values:

h	3	4	5
C_m	0.3034	0.3020	0.3014

$Q = cfs$, Q_m = cu. ft. per min., h = head on notch, ft.

Angle at vertex 54°, area of orifice is half that for 90°; C_m is slightly greater than half C_m for 90°.

Prevention of inward flow at sides of notch, due to narrowness of channel or roughness of upstream face of weir, increases discharge.—(James Barr, Glasgow Univ., 1910.)

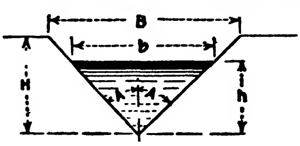


FIG. 340.—Triangular Weir.

Tests at Cornell University: For perfect contraction, with sharp edges,

$$Q = \frac{4}{15}\,C\,\frac{B}{H}\sqrt{2g}.h^{\frac{5}{2}} = \frac{8}{15}\,C \tan A\sqrt{2g}.h^{\frac{5}{2}}$$

B = width of notch at hight H

If $K = \frac{4}{15}\frac{B}{H}\sqrt{2g}$, a constant for a given weir, then $Q = Kh^{\frac{5}{2}}$ theoretically, or $Q = CKh^{\frac{5}{2}}$, where C = coef. of contraction.

Table 195. Triangular Weirs. Coefficient of Contraction

2A	Variation of head, ft.	Readings, number	Range of C	C* Average
28°	0.4775–3 0375	17	0.586–0.607	0.5994
60°	0.2303–2 9922	24	0.435–0.587	0.5645
90°	0 440 –2.458	15	0.532–0.591	0 5705
120°	0.294 –1 501	13	0 574–0.603	0 5953

* If one ignores heads below 0 5′, $C = 0.6002$, 0.5752, 0.5752 and 0.5966 for 28°, 60°, 90° and 120°, respectively.

From these data, experimenter R. B. Daudt concludes: (1) For heads between 0.5 and 3 ft., coef. of contraction is fairly steady. Average C might be within 1 per cent. of truth. (2) Knifelike edge of weir secured almost perfect contraction, accounting for lower values of C than obtained in other experiments. (3) Intermediate weirs between 28° and 120° have lower coefficients than these limits.—(Cornell "Civil Engineer," Mar., 1912.)

Trapezoidal Weirs. Cippoletti's formula is $Q = 3.36\frac{2}{3}\ LH^{\frac{3}{2}}$, and applies to a weir with end slopes 4 on 1, sill horizontal, all edges sharp; the plane of the weir should not be more than 4° from a perpendicular to the axis of the canal; other conditions are the same as for Francis weir with full contractions. For lengths 3, 4, 5, 6, 7, 8 and 9 ft. of Cippoletti weir, Flinn and Dyer in 1893,* found an agreement with the standard Francis weir, within 0.2 to 5 per cent for heads from 0.2 to 1.4 ft. The experiments were made at the Holyoke testing flume.

J. C. Stevens experimented on 6-in., 2-ft., and 3-ft. weirs. The discharge curves gave continually larger results than indicated by the Cippoletti formula. These differences result from two causes: (1) On long weirs, the effect of end contraction, a curvilinear rather than a rectilinear function, is negligible, whereas it is a larger item on short weirs; end slopes should be greater than 4 on 1 properly to compensate for contraction. (2) The slightly rounded crest reduced the bottom contraction. The weir tested had 4 on 1 end slopes. It was made of 2-in. plank cut to shape, and beveled 45° on the downstream side. A galvanized iron plate in one piece protected the edge.—(J. C. Stevens, E. N., Aug. 18, 1910, p. 171.)

Flow of Water over Dams. From Cornell experiments on flow over weirs, Gardner S. Williams has deduced coefficients in Table 196 to be applied to Francis or Bazin weir formula (based on sharp-edged weir) in computations for discharge over dams.

Table 196. Discharge Coefficients for Overflow Dams

Type†	A	A	A	A	A	A	A	A	B	B	C	D	E	F
b(feet)	0 48	0.93	1 65	3 17	5 89	8 98	12 24	16 30	6 65	11 25			.	..
Head (ft.)														
0 5	0.902	0.830	0.819	0 797	0 785	0.783	0.783	0.783	1.060	1.060	0 968	0 971	0.971	0 971
1 0	0 972	0.904	0.879	0 812	0.800	0.798	0.795	0.792	1.079	1 079	1.008	1.040	1 040	0 983
1.5	1.000	0.957	0.910	0.821	0 807	0 803	0.802	0.797	1 091	1 092	1.032	1.083	1.092	1 022
2.0	1.000	0.989	0.925	0.821	0 805	0 800	0.798	0.795	1.086	1.097	1.041	1.105	1.126	1 040
2.5	1.000	1 000	0 932	0.816	0.800	0 795	0.792	0.789	1.076	1 096	1 043	1.118	1.146	1 057
3 0	1 000	1.000	0.938	0 813	0 796	0 791	0.787	0.784	1 067	1.095	1 044	1.128	1.163	1 072
3 5	1.000	1.000	0.942	0.810	0.793	0 787	0 783	0 780	1 060	1 094	1 045	1.136	1.177	1 085
4 0	1 000	1 000	0 947	0 808	0 790	0 783	0.780	0 777	1 054	1 093	1.046	1 144	1 190	1.097

*T. A. S. C. E., Vol. 32, 1894. † See page 602.

Bazin's formula:* $Q = \left(0.405 + \frac{0.00984}{h}\right)\left(1 + \frac{0.55h^2}{(h+p)^2}\right)Lh\sqrt{2gh}$

where Q = discharge, cu. ft. per sec.; h = observed head, feet; p = hight of crest above channel, feet; L = length of weir, feet.—(E. N., Jan. 12, 1911, p. 38.)

From measurements of flow the following rating formulas have been determined.

$Q = 2.491\ Lh^{1.77}$	Merced dam
$Q = 3.088\ Lh^{1.50}$	La Grange dam† (Turlock)
$Q = 3.110\ Lh^{1.65}$	Yakima dam
$Q = 3.196\ Lh^{1.59}$	Old Austin dam (Texas)‡

(Ref: W. F. Martin, E. N., Sept. 29, 1910, p. 321.)

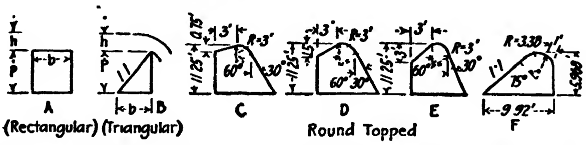


FIG. 341.—Types of crests referred to in Table 196, p. 601.

Coefficients of Discharge for Flow through Notches. In constructing dams, it is often necessary to approximate the number of openings to be left to take the river flow. Tests were made at McCall's Ferry dam, with rectangular openings 45 ft. deep, and 40 ft. wide, left in the concrete, 7 ft. above the river bed; each notch, or opening, was about 40 ft. long (= thickness of dam). Two formulas were tested for constants: (1) Fteley and Stearns:

$$Q = C_sL(H + \tfrac{1}{2}H_t)(H - H_t)^{\frac{1}{2}}$$

(2) Ordinary weir formula:

$$Q = C_wL(H - H_t)^{\frac{3}{2}}$$

Q = discharge, cfs.
H = measured hight, ft., of upstream pond level over bottom of notch, corrected for velocity of approach.
H_t = measured hight, ft., of tail water below dam over bottom of notch, corrected for velocity of retreat.
L = crest length, ft., corrected for end contraction = $40 - 0.2\,(H - H_t)$.
C_s = coef. of discharge for submerged weir.
C_w = Bazin's coefficient, using head, $H - H_t$.

H varied from 14.2 to 24.4 ft., with a corresponding variation in velocity H_t from 4.40 to 7.60 ft., and in Q from 4550 sec. ft. to 10,100 sec. ft. C_s varied from 2.35 for first condition, to 2.47 for H = 17 and 18 ft. C_w varied from 3.92 to 4.01 corresponding to heads and quantities first mentioned.—(Cornell Civil Eng., May, 1910.)

* See page 597 for sharp-edged weir.
† Profile on page 179.
‡ Profile on page 174.

Discharge through Submerged Orifices. Sixty-three experiments. At the sides of a long horizontal wooden channel were screwed a pair of vertical posts, held together at the upper end by a yoke in which moved a screw attached to a gate-board. To cause the opening to be submerged, a dam was built in the channel below. Formulas for coefficients of discharge:

$$C = \frac{Q}{ab\sqrt{2g(h_1 - h_2 + k)}}$$

h_1 = upstream head on top of orifice
h_2 = downstream head on top of orifice
a = width of orifice
b = channel width
k = constant.

The formula giving the best results is:

$$C = \alpha + \beta\sqrt{\frac{a}{h_1 + \frac{a}{2}}} + \gamma\left\{\frac{1}{\left(h_2 + \frac{a}{2}\right)}\sqrt{b}\right\} \qquad \text{(A)}$$

in which $\alpha = 0.435$, $\beta = 0.257$, $\gamma = 0.031212$. Probable error = 0.0256.

The following formula is about as accurate and much simpler:

$$C = 0.541 + 0.150\frac{\sqrt{a}}{h^2\frac{a}{2}}. \quad \text{Probable error} = 0.0276.$$

In Weisbach's 14 experiments the channel was 1.2 ft. wide, and had no obstructions at gate to form side contractions, as in Bornemann's, opening at gate having full width of channel; gate-board had sharp bottom edge. Experiments were made with three hights of gate opening, 0.12, 0.19 and 0.25 ft. Head above top of opening varied, on upstream side from 0.29 to 0.90 ft., and on downstream side from 0.15 to 0.44 ft. Discharge varied from 0.36 to 1.28 cu. ft. per sec.; C varied from 0.689 to 0.919. Formula (A) fits these experiments well when $\alpha = 0.3586$, $\beta = 0.8603$ and $\gamma = 0.002618$. Boileau made three experiments with opening at gate = full width of channel = 2.9 ft., gate opening = 0.33 ft. Coefficients of discharge were from 0.02 to 0.04 less than those of Weisbach.—(Bornemann. Der Civilingenieur, Vol. 26, 1880.)

Flow of Liquids through Cracks.* Results of test with filtered water check well with results from formula in Lamb's "Hydrodynamics." Bronze piston, 6 in. diam., fitted in cylinder 8 in. long with clearance of 0.0018 in. Water temperature, 74° F.

Table 197. Flow of Water through Cracks

Pressure, lbs. per sq. in.	Flow, cu in per sec.	
	Calculated	Measured
20	0 21	0 14
40	0 42	0 35
60	0 62	0 61
80	0 83	0 83
100	1 04	1 04
120	1 25	1 28

* Ernst Jonson, E. N., Dec. 19, 1912.

Flow through Submerged Screens. An investigation on 12-in. square screens, made up of 3-in. by ½-in. pine slats, 1-in. centers, with different conditions of pointing, showed that the flow closely approaches flow through short tubes. $Q = C_o A \sqrt{2g(H_v)}$. A = open area of screens, sq. ft. H_v = difference in water levels above and below screens diminished by the difference in velocity heads. For screen with square corners, C_o = 0:811; with downstream face pointed, C_o = 0.832; with upstream face pointed, C_o = 0.985; with both stream faces pointed, C_o = 1.032. Tests on submerged perforated screens, 12 in. square and 0.30 in. thick, gave coefficients corresponding to the flow through orifices in thin plates. With ½-in. holes, spaced 2½ in. on centers, C_o = 0.640. With ½-in. holes, spaced ¾ in. on centers, C_o = 0.758.—(Henry Ryon, "Cornell Civil Engineer," Dec., 1910.)

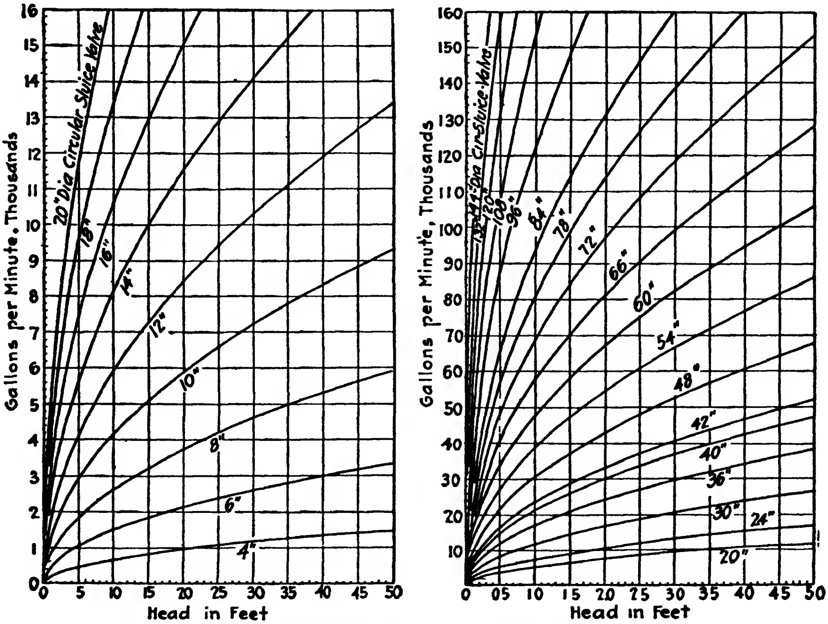


FIG. 342.—Discharge through circular sluice gates.

Gates and Valves.—*Loss through Reducer and Gate.* The loss through a 24-in. gate valve, with conical sections on each end, connecting it to the 42-in. wood-stave pipe line at Atlantic City, is about 0.11 ft., when carrying 5.5 mgd. by test.—(Kenneth Allen, Jl. N. E. W. W. Assn., Dec., 1904).

Loss through Coffin Sluice Gates. (Prepared by Freeman C. Coffin, M. Am. Soc. C. E.) Diagrams (Fig. 342) are not designed as a basis for the measurement of water in any other way than as an approximation. Sluice gates are set under a great variety of conditions; sometimes flat on the face of a thick masonry wall, when the conditions of a short tube are approximated; at other times flat on a thin wall where the conditions are nearer those of an orifice; again, on the end of a pipe, and the actual coefficient of discharge perhaps ranges from 0.60 to 0.80. The coefficient used, 0.67, suits the varying condi-

tions as well as any uniform coefficient can. These diagrams indicate the size of gate which can be used with safety; the discharge through the given gate opening under the specified head; and the loss of head through the given gate opening for a specified discharge. Refinements and close estimations of discharge or loss of head are not required, as the regular sizes of sluice gates vary by a considerable margin, and it is wise when in doubt to use the larger size. For heads and quantities not given exactly in the diagram, if straight-line interpolation is not sufficiently accurate, use following methods: (1) Where the head is given and the discharge required, find the discharge due to an assumed head in the diagram, and use the formula: $\left(\frac{\text{given head}}{\text{assumed head}}\right)^{\frac{1}{2}}$ $\times$ discharge due to assumed head = required discharge. What is the discharge of a 12-in. by 16-in. sluice gate with a loss of head of 7 ft.? Assume a loss of head of 10 ft. and get corresponding discharge from the gate = 10,160 gal. per min. Then, $(\frac{7}{10})^{\frac{1}{2}} = 0.836$. $0.836 \times 10{,}160 = 8500$ gal. per min. (2) When the discharge is given and the loss of head required, find the loss due to an assumed discharge given in the diagram and use the formula $\left(\frac{\text{given discharge}}{\text{assumed discharge}}\right)^2 \times$ loss of head due to assumed discharge = loss of head for given discharge. What is the loss of head due to a discharge of 35,000 gals. per min. through a 48-in. circular sluice gate? Assume the discharge for a loss of head of 2 ft., namely, 43,000 gals. Then $\left(\frac{35000}{43000}\right)^2 = 0.667$; $0.667 \times 2 = 1.334$ ft., loss of head for a discharge of 35,000 gals. See also p. 424.

Flow in Capillary Tubes (Hazen).† $V = csd^2\left(\frac{t+10}{60}\right)$; c being a coefficient (average value = 495), s the slope, d the diam. in inches and v = velocity, ft. per sec. t is temperature in degrees F. The factor $\left(\frac{t+10}{60}\right)$ was deduced from experiments at Lawrence to represent the flow of water, at different temperatures, through capillary tubes. It also represents the relative rates at which water passes through sand, and at which silt settles through quiet water. This term is more convenient for practical use than the conventional factor of Poiseuille (see p. 79) and is sufficiently accurate.

† T. A. S. C. E., Vol. 51, 1903, p. 317.

Table 197a. Discharge and Loss of Head, Coffin Rectangular Sluice Gates

Size of gate, width, hight	Area, square feet	Loss of head in feet*																	
		0.1	0.25	0.50	0.75	1	2	3	4	5	6	8	10	15	20	25	30	35	40
		Discharge in gallons per minute																	
In																			
8×8	0.448	341	537	763	933	1,081	1,528	1,867	2,158	2,412	2,640	3,050	3,414	4,165	4,820	5,370	5,900	6,380	6,820
10×10	0.694	529	835	1,180	1,448	1,670	2,365	2,890	3,340	3,735	4,090	4,720	5,288	6,470	7,470	8,350	9,140	9,870	10,270
12×12	1.000	762	1,208	1,709	2,090	2,413	3,430	4,180	4,830	5,400	5,910	6,830	7,622	9,315	10,780	12,030	13,210	14,280	15,270
12×16	1.333	1,016	1,630	2,270	2,780	3,216	4,530	5,540	6,390	7,150	7,810	9,030	10,160	12,420	14,350	16,300	17,560	19,000	20,250
12×20	1.666	1,270	2,080	2,840	3,480	4,020	5,690	6,960	8,040	8,990	9,840	11,350	12,697	15,520	17,950	20,800	22,000	23,780	25,400
16×16	1.777	1,354	2,133	3,025	3,700	4,288	6,050	7,420	8,570	9,570	10,480	12,100	13,544	16,500	19,020	21,330	23,400	25,220	27,000
16×20	2.222	1,694	2,670	3,780	4,640	5,362	7,575	9,275	10,700	11,970	13,100	15,130	16,936	20,700	23,900	26,700	29,250	31,600	33,700
16×24	2.666	2,032	3,230	4,550	5,575	6,433	9,120	11,150	12,880	14,400	15,750	18,200	20,320	24,900	28,720	32,300	35,250	38,000	40,700
18×18	2.250	1,715	2,700	3,825	4,680	5,429	7,670	9,380	10,820	12,100	13,250	15,300	17,149	20,900	24,120	27,000	29,600	31,900	34,200
18×24	3.000	2,287	3,625	5,120	6,270	7,239	10,230	12,520	14,440	16,180	17,700	20,480	22,868	27,950	32,300	36,110	39,500	42,700	45,600
18×36	4.500	3,430	5,400	7,640	9,350	10,859	15,280	18,700	21,600	24,100	26,400	30,650	34,300	41,750	48,300	54,000	59,000	63,800	68,100
20×20	2.777	2,117	3,340	4,730	5,800	6,701	9,450	11,590	13,380	14,920	16,330	18,900	21,167	25,850	29,900	33,400	36,600	39,500	42,300
20×24	3.333	2,541	4,010	5,675	6,960	8,043	11,380	13,920	16,040	17,970	19,670	22,700	25,405	31,000	35,750	40,100	43,900	47,500	50,800
20×36	5.000	3,811	6,050	8,550	10,480	12,066	17,090	20,900	24,130	26,950	29,550	34,200	38,113	46,750	54,000	60,500	66,200	71,500	76,500
24×24	4.000	3,049	4,800	6,800	8,330	9,652	13,600	16,680	19,250	21,500	23,580	27,200	30,490	37,200	42,900	48,000	52,600	56,850	60,800
24×36	6.000	4,574	7,220	10,180	12,480	14,479	20,350	24,930	28,870	32,300	35,350	40,800	45,736	55,800	64,400	72,100	79,000	85,200	91,000
24×48	8.000	6,098	9,620	13,600	16,500	19,305	27,250	33,400	38,600	43,200	47,300	54,600	60,981	74,400	86,000	96,200	105,200	114,000	121,800
24×60	10.000	7,623	12,080	17,090	20,900	24,131	34,250	41,800	48,300	54,000	59,200	68,300	76,227	93,300	107,800	120,800	132,200	142,800	152,800
24×72	12.000	9,147	14,425	20,500	25,000	28,957	40,900	50,100	57,800	64,600	70,800	81,600	91,472	111,800	129,000	144,250	158,000	170,800	182,800
24×96	16.000	12,196	19,200	27,180	33,250	38,610	54,400	66,700	77,100	86,200	94,400	109,000	121,963	148,700	171,900	192,000	210,000	226,800	242,800
24×120	20.000	15,245	24,150	34,200	41,800	48,262	68,300	83,800	96,800	108,000	118,200	136,800	152,454	186,800	216,000	241,500	264,000	285,300	305,000
30×30	6.250	4,764	7,520	10,630	13,020	15,081	21,280	26,050	30,100	33,700	36,800	42,500	47,640	58,100	67,200	75,200	82,200	89,000	95,000
30×36	7.500	5,717	9,000	12,730	15,620	18,098	25,500	31,250	36,200	40,400	44,200	51,000	57,169	69,800	80,800	90,000	98,600	106,500	114,000
30×48	10.000	7,623	12,080	17,090	20,900	24,131	34,200	41,800	48,300	54,000	59,200	68,300	76,227	93,300	187,800	120,800	132,100	142,900	152,800
30×60	12.500	9,528	15,200	21,300	26,100	30,165	42,600	52,200	60,300	67,300	73,800	85,200	95,283	116,800	134,700	150,800	165,000	178,000	190,300
30×120	25.000	19,057	30,100	42,500	52,200	60,328	85,200	102,450	120,800	134,800	147,500	170,300	190,568	232,800	268,800	301,000	329,000	356,000	380,000
36×36	9.000	6,860	10,800	15,250	18,750	21,718	30,600	37,500	43,300	48,400	53,000	61,300	68,604	83,800	96,700	108,000	118,500	128,000	136,800
36×42	10.500	8,004	12,620	17,880	21,880	25,337	35,800	43,800	50,600	56,500	62,000	71,700	80,037	98,000	113,000	126,300	138,300	149,500	159,000
36×48	12.000	9,147	14,425	20,450	25,000	28,957	40,900	50,100	57,800	64,600	70,800	81,600	91,472	111,800	129,000	144,250	158,000	170,800	182,000
36×60	15.000	11,434	18,030	25,520	31,300	36,197	51,200	62,800	72,500	81,000	88,700	102,500	114,341	139,800	161,500	180,300	197,800	213,500	228,000
36×96	24.000	18,293	28,800	40,800	50,000	57,915	81,800	102,000	116,000	129,800	141,800	164,000	182,925	223,500	258,000	288,000	317,000	342,000	365,000
40×40	11.111	8,469	13,500	18,900	23,200	26,812	37,900	46,400	53,700	60,000	65,700	76,000	84,694	103,300	119,000	133,500	146,200	158,000	169,000
40×60	16.666	12,704	20,100	28,400	34,800	40,216	56,800	69,600	80,400	89,900	98,400	113,500	127,038	155,800	179,800	201,000	220,000	237,900	254,000
42×42	12.250	9,338	14,730	20,880	25,600	29,560	41,800	51,300	59,200	66,200	72,400	83,800	93,376	114,200	132,000	147,300	161,000	174,000	186,000
42×60	17.500	13,340	21,000	29,720	36,500	42,229	59,500	72,900	84,300	94,200	103,200	119,200	133,396	162,800	188,000	210,000			
42×72	21.000	16,008	25,200	35,700	43,700	50,675	71,800	87,500	101,000	113,000	124,000	143,200	160,077	195,000	225,500	252,000			
48×48	16.000	12,194	19,200	27,180	33,250	38,610	54,300	66,700	76,900	86,000	94,300	109,000	121,943	148,800	171,800	192,000			
48×60	20.000	15,245	24,000	34,000	41,750	48,262	68,100	83,500	96,500	107,900	118,200	136,800	152,454	186,000					
48×72	24.000	18,294	28,850	40,800	50,000	57,915	81,500	100,000	115,700	129,000	141,500	163,500	182,944						
48×96	32.000	24,393	38,350	54,200	66,600	77,220	108,800	133,300	154,000	172,000	188,800	218,000	243,926						
60×60	25.000	19,057	30,000	42,500	52,200	60,328	85,100	104,200	120,300	134,800	147,500	170,300	190,568						

PART VI
MATERIALS AND MISCELLANY

CHAPTER XXVI

MASONRY AND PUDDLE

CONCRETE

Solubility of Cement. Experiments at the laboratory of the Catskill aqueduct, N. Y., showed that Portland cement in lean concrete through which fresh (Croton) water had percolated for nearly 2 yrs. was dissolved to such extent that the strength of the concrete was only 20 to 35 per cent. of normal for concretes of the given age and mixture. The sand was standard crushed quartz and the coarse aggregates included glass at one extreme of a series of specimens and a soluble limestone at the other. The aggregates were unaffected and seemed to have had no effect on the result. Specimens through which the greatest quantity of water had passed were weakest. The proportions of the concretes were 1:3½:6.

Sand* for mortar or concrete in hydraulic structures may be coarse, medium or fine, but must not have grains of uniform size nor a large proportion of substantially one size; a gradation from coarse to fine which has minimum voids is best. Grains should be hard, durable material, with not more than 3 per cent. organic matter or 5 per cent. clay. Dirty sand should be washed; too uniform sand should be mixed with other. Some sands apparently clean will not make good mortar, some ingredient interfering with the setting of cement; less than 1 per cent. is sometimes troublesome. Sand which it is impossible

Table 198. Standards for Classification of Gravels and Sands by Sizes†

Conventional name	Corresponding diam.	
	mm.	in.
Coarse gravel	50.0 to 5 0	1 97 to 0 20
Fine gravel	5.0 to 1 0	0 20 to 0 04
Coarse sand	1 0 to 0 5	0 04 to 0 02
Medium sand	0 5 to 0 25	0 02 to 0 01
Fine sand	0.25 to 0 10	0 01 to 0 004
Superfine sand	0 10 to 0 05	0 004 to 0 002
Rock flour	0 05 to 0 01	0.002 to 0 0004
Superfine flour	0 01 to 0 005	0 0004 to 0 0002
Clay	0.005 to 0 0001	

* See E. R. June 12, 19 and 26, 1915, articles by C M Chapman and N. C. Johnson.
† Burr-Hering-Freeman, "Report of Commission on Additional Water Supply for the City of New York," 1903.

to use with one brand of cement will sometimes give satisfactory results with another brand. Some mortars and concretes which harden very slowly ultimately become very strong. Best determination is to make large test cylinders or cubes of concrete or mortar with the sand and cement which it is expected to use. For cylinders 8 in. in diam., length 16 in. is standard.

Sizes of Separation of Screens for Sand. By size of separation is meant the diam. of a sphere equivalent in volume to the grain which will barely pass through. Due to inaccuracies of making, any sieve may vary 15 per cent. from above. Every sieve should be rated.

Proportioning Concrete.* The "Ideal curve" of Taylor and Thompson† as a guide for concrete on hydraulic work is limited in application because: (1) the percentage of cement has often been fixed by specification irrespective of the character of the aggregates (it should not be); (2) there are few places where more than one source of sand and stone is available, allowing small choice in aggregates; (3) grading material by screening the stone into several sizes is generally impracticable. The proportion of sand determined by the "ideal curve" must be increased to get a plastic mix. Where water-tightness is of primary importance, proportion the concrete so that the mortar is in excess of the voids in the large aggregate by 15 to 20 per cent. (sufficient to make the mixture plastic and to compensate for irregular mixing and placing). The density test is the most scientific method of determining proportions. Make up trial mixtures of the materials to be used, according to the consistence intended. The materials are carefully weighed and mixed, and a batch placed in an 8-in. wrought-iron pipe 10 in. long capped at one end, and the hight of concrete noted. Several batches should be tried in which the weights of cement and water, and the total weight of aggregates remain constant, but the proportion of sand to stone is varied. That proportion is densest which gives the smallest volume of concrete. Use the proportion nearest this, which at the same time looks well and works smoothly. Measurements on construction work show a tendency to run up the proportion of sand, as this facilitates mixing and placing.

Tufa Cement. Sometimes tufa is calcareous deposit, but that described here is of volcanic origin. It is of grayish or creamy color, and of low specific gravity in rock form. As tufa is light and easily crushed, the grinding machinery is cheap. Tests and field experience on the Los Angeles aqueduct, show the tufa cement to be as strong and as satisfactory generally as straight cement when used in concrete, and very much cheaper as tufa deposits were found along the line. When mixed with lime it makes a strong hydraulic cement. Generally, it was used in equal volumes with Portland cement, mixed by regrinding together.—(J. B. Lippincott, T. A. Soc. C. E., Vol. 76, 1913.)

Durability of Concrete. In 1848 the Erie Railroad built Starrucca viaduct, using over 1000 cu. yd. of 1:1.5:3.5 Rosendale cement concrete. F. L. Stuart, Chief Engineer, stated that an examination in 1907 showed the concrete in good preservation. The Department of Docks and Ferries, New York City,

* See E. R. Jan. 23, Feb 6, 13 and 27, and Mar. 6 and 13, 1915, articles by N. C. Johnson.
† "Concrete, Plain and Reinforced," p. 776, 1909.

has built over 7 mi. of Portland cement sea wall, some of it previous to 1875; it is in good condition, except for about 2.5 ft. near low-water mark, where the waves and ice have eroded it slightly. In 1907 D. K. Colburn, Bridge Engineer of Galveston, Harrisburg and San Antonio Railway Co., had occasion to remove the tops of concrete piers built in Medina river in 1881; the concrete was found in perfect condition. It was composed of a 1:1.66 mix of Portland cement and irregular flinty pebbles.

Erosion of Concrete: Permissible Velocity of Water in Contact. South canal, Uncompahgre Valley, carries 300 sec.-ft. at 20 ft. per sec. No erosion occurred in one season; slimy, mosslike accumulations are appearing under the water. On Strawberry project, Provo, Utah, velocities of 40 ft. per sec. have been resisted for 2 yrs. by a spillway on a 42 per cent. slope. Turlock dam, see p. 179, is submerged by water laden with sand, with a drop of about 100 ft.; 18 yrs.' wear, under velocities as high as 70 ft. per sec., has had slight effect on the concrete at the base; erosion has been greatest on the sand and cement, leaving the greenstone aggregate projecting in spots; ½ in. is the maximum wear. The wear can probably all be laid to the sand in the water. A turbulent stream discharging under 60-ft. head at Roosevelt dam, through an unlined tunnel, did considerable damage. After discharging water for 6 months through the Pathfinder dam at velocities of 75 to 90 ft. per sec., marks of the forms still showed plainly on the concrete lining; a turbulent stream through a conduit of nearly double this cross-section, under the same head, injured the lining. Observations show: (1) that concrete is not injured by clear water gliding over it at high velocities, if the velocity or direction are not abruptly changed; (2) that concrete subjected to the impact of water at high velocities is rapidly eroded. (A. P. Davis, E. N., Jan. 4, 1912.) A water-wheel nozzle, lined by Wm. Mulholland, Los Angeles, with Portland cement mortar to reduce the diam. showed all the marks of the wooden mandril about which the mortar was placed, after several years' use. Tests by E. C. Clark (Trans. Am. Soc. C. E., Vol. 14, 1885, p. 167) indicated that 1:2 Portland cement mortar is the best proportion for resisting abrasion, the resistance being nearly double a 1:1 or 1:2.5 mixture; for natural cements, 1:1 gave the best results. Concrete sewer inverts in Duluth, Minn., showed no deterioration after 20 yrs., while brick sewers, built at the same time, had to be renewed in 6 or 7 yrs. One drain, 4 ft. diam., 2000 ft. long, on a 13 per cent. grade (velocity 42 ft. per sec.) had an invert of flat granite flags, laid with 1:1 Portland cement mortar. There was a heavy storm flow, carrying sand, etc. After 2 yrs.' wear, the ridges at the joints indicated that the mortar was more durable than the granite.—(Taylor and Thompson, Concrete Plain and Reinforced, pp. 125 and 681.)

Permeability of Concrete. At laboratory of Catskill aq., N. Y., experimental studies were made of the permeability of concrete. Permeability is subject to accidental variations in a much greater degree than strength, yet a study reveals certain governing laws. The quantity of cement used is important. Concrete of properly graded natural aggregates containing over 15 per cent. of cement by weight, was found to be practically impermeable under heads not exceeding 200 ft. As the percentage of cement was reduced,

permeability increased slowly until about 11 per cent. was reached, when the permeability increased rapidly. Proper grading of the aggregates is another important condition. High percentages of fine particles produce a decided reduction in permeability, while tests with natural sand, of which 25 per cent. was finer than 0.01 in. and 10 per cent. finer than 0.006 in. in diam., in a 1:3 mortar, resulted in practical impermeability. When made from broken stone and screenings, concrete is more permeable than when made from gravel and natural sand. Concrete is more permeable in a direction parallel to its bed than perpendicular thereto. The dense skin formed on the surface of the concrete was found to have a very important effect on permeability. Leakage through specimens 6 in. thick, from which the surface skin had been removed, was much greater than through specimens otherwise similar, the surface of which was undisturbed. Studies were made of the effects of various materials incorporated in the matrix of the concrete in reducing permeability, including clays, hydrated lime and puzzolan and sand cements, these two cements being more finely ground than Portland. The conclusion was reached that concrete, practically impervious under a head of 200 ft., could be produced with any of these materials, but that equally good results can be secured at no greater cost by using sufficient Portland cement, the latter method having the advantage of increasing the strength, while other materials usually reduce it. Another very important detail is to keep the concrete thoroughly wet from the time it begins to harden until it is about 2 weeks old, and to protect it from the hot sun. Concrete should not merely be lightly sprinkled occasionally, but thoroughly wet, and not permitted to dry out for about the period named, depending upon temperature, humidity, thickness of mass and other local conditions. Prolonged and thorough mixing is important, but even more important is scrupulous care in placing so as to avoid all porous or honeycombed spots.

Tests on lean concrete mixes for drainage blocks showed that no concrete of sufficient strength to be handled in blocks, can be depended upon to be permeable enough to act as a free drain. Horizontal stratum of relatively impermeable concrete is formed at the bottom of the block mold, which should be chipped off to obtain the greatest permeability.

Waterproofing Concrete. See also under Tanks, pp. 523 and 524. *Rich Concrete.* Permeability may be decreased by proper mixing. The richest concrete up to a certain point shows the least permeability; mixes richer than 1:3 are liable to crack. Permeable concrete tends to become tight after the passage of water through it. The mix should be such as to secure dense concrete with excess of mortar. Thorough mixing produces a homogeneous mass. Wet mixes are tighter than dry, if not excessively wet. 1:3 to 1:6 mixtures have given satisfactory results.

Surface Coatings. Rich surface coatings of 1:1 or 1:2 mortar are often applied to horizontal or inclined surfaces. Concrete should be surfaced while green and the coating troweled in place to produce a thickness of ½ in. to 1 in. Heat causes crazing. Such coatings may be good for submerged surfaces of reservoirs. Plastering on concrete, however, is rarely successful unless done by specially skilled workmen with faithful attention to numerous details.

Surface Skin. F. F. Moore, Jl. Assn. Eng. Soc., Sept., 1911, gives some test results. When concrete is cast against smooth metal forms, a surface skin is produced, which, compared to the body of the concrete, is highly resistant to the passage of water. Laboratory tests indicated permeability under water pressure of 1 :2.6 :4.9 concrete, 6 in. thick, with the skin intact, to be only 1.2 per cent. of that of the same concrete without surface skin; as a resistant, the skin is equal to many feet of concrete. With richer concrete this proportion would be reduced; nevertheless water-tightness depends largely on the continuity of the skin. Sand gave more uniformly dense concrete than crushed stone screenings. Surface skin produced by troweling concrete as it sets is a great help toward waterproofing. Pittsburgh filtered-water reservoir floor is two layers of concrete each 4 in. thick, in blocks lapping 3 in., each surface being troweled hard. This reservoir holds 25 ft. of water without measurable leakage although the floor rests directly upon loose sandy gravel, extremely pervious.

Sylvester's "Process for Repelling Moisture from External Walls" has been used some time in England; it was used in United States in 1870 on the Central Park gate-house (Trans. Am. Soc. C. E., Vol. 1, 1872, p. 204). This process consists of 2 coats, the first a solution of castile soap in water ($\frac{3}{4}$ lb. soap per gal.); the second, an alum solution ($\frac{1}{8}$ lb. alum per gal. of water). A complete solution is necessary. Surfaces to be treated should be cleaned. At the time of application, air temperature should be above 50° F. Soap coatings should be boiling hot when applied. Use a flat brush; care should be taken to prevent frothing on the masonry. First coating should stand 24 hrs. and become hard and dry before putting on the alum coating. The latter may be at a temperature of 60° or 70°; it should stand 24 hrs. before the second application of soap. Soap and alum combine to form insoluble compounds, penetrating the pores of the concrete. Several coats are generally needed. Aluminum sulphate* (popularly called "alum") is cheaper than alum, and only two-thirds as much is required.

The advantages of using a rosin soap are both chemical and economical. The compounds precipitated are more stable and insoluble, and the rosin soaps are cheaper. Babbitt's laundry soap would answer and costs but $4 per cwt. A wash of alum, lye and cement has been applied on fortifications with good results. (Report Chief of Engineers, 1902, p. 2482.) U. G. Hayne reports a tank under 6-ft. head, treated with 2 external and 2 internal coats, water-tight 7 yrs. after building. Coats were placed on a layer of 1 cement to 2 stock mortar. A complete solution was made in proportion of 1 lb. concentrated lye, and 5 lbs. of alum to 2 gals. water. Heating aids solution and does not affect ingredients. To this "stock solution" add 10 lbs. of cement per pint, thinning with water (about 11 gals. per pint) to a consistence easy to spread. Use a flat brush. A good mixture lathers freely. Surfaces to be treated should be not over 3 or 4 days old and not too dry; they should not be exposed to the sun at time of application. If the wash is applied too thick, it is liable to flake off. This wash has not proved satisfactory on old work.—(Reid's "Concrete and Reinforced Concrete Construction, 1908.")

Concrete Paints. Linseed oil paints should not be applied directly to new

*Am. C. E. Pocketbook, p 525 (Wiley).

concrete or any concrete which has not been exposed to weather for a time. Free lime, not thoroughly weathered out, will saponify the oil, destroying its adhesive power, and the paint will soon scale off. As it is difficult to determine when free lime is out of concrete, it is safer to avoid linseed oil paints, in direct contact with concrete. Chas. MacNichol, in Trans. Am. Soc. Testing Materials, 1910, describes his method: After concrete has thoroughly set, give it a priming coat of a solution of zinc sulphate, 8 lbs. to a gallon of water, applied with a brush. After priming coat is dry, linseed oil may be put on. Several years' use with satisfactory results are claimed. For all paints, an alkali-proof size is desirable.—(Cement Age, Feb., 1911.)

Incorporated Waterproofing. I. O. Baker (Conc. Eng., Feb., 1910) says that the safer practice is to dissolve both the alum and soap in the mixing water, 1 part alum to 2 parts hard soap by weight—any reasonably pure soap. As not more than 3 per cent. hard soap can be dissolved in cold water, the alum is limited to 1.5 per cent. of water weight. Prof. Baker also gives the following formula in his "Masonry Construction:" 1 per cent. by weight of alum is added to dry cement and sand; 1 per cent. potash soap or ordinary soft soap is dissolved in the mixing water. An insoluble compound results. W. K. Hatt has successfully used 5 per cent. solution of alum and 7 per cent. solution of soap, in equal parts in mixing concrete. Many patented compounds are on the market, claiming that by addition of 1 to 2 per cent. by weight to cement, permeability is reduced. Many of these, by test, have proved useless or worse; chemical analyses show some ingredients to be impermanent and others injurious to concrete. Intelligent selection of aggregates, skillful, thorough mixing, careful placing, and subsequent protection from wind and sun, and keeping wet for about 2 weeks., are the best means for making concrete water-tight. (Much of the preceding on Waterproofing is from "A Treatise on Concrete, Plain and Reinforced," Taylor and Thompson, 1909.)

CYCLOPEAN MASONRY

To procure good work, following conditions are essential: Concrete should be mixed very wet and should always be dumped in a low spot on the dam or wall so that the water is confined and cannot drain out. If racking is used, small rubble walls should be built to confine the concrete. Fine aggregate for concrete should not be composed entirely of rock screenings, as this does not make a well-graded mix, and forms a concrete which is "bony," making it hard to bed the large stones; such concrete does not retain its fluidity. A suitable quantity of well-graded natural sand should be added to the screenings. Concrete should be mixed in batches and should be placed as soon after mixing as possible, otherwise it settles in the bucket, and is hard to dump. Batches larger than 2 yds. are hard to work over and spread. Care should be taken to avoid laying stones with concave beds or long flat projections, such stones being hard to settle into the concrete, and likely to entrain air and water. In Barren Jack Dam, Australia, bond of cyclopean masonry was increased by a system of units, cruciform in shape, with area of 1080 sq. ft. These units vary in hight from 9 to 15 ft., being so arranged that continuous units break joint

both horizontally and vertically.—(E. R. May 6, 1911, p. 497.) (See also p. 161 for other construction experience.) List on p. 166 includes some cyclopean masonry dams.

Weight of Masonry in Dams. Church and Fteley, in their report on Quaker Bridge dam, assume weight of granite masonry at 156.25 lbs. per cu. ft.; the Board of Experts on this dam assumed 146.25, citing Krantz as using 143.7, and Bouvier, 147.3. Records from Boyd's Corners dam gave 146.0 lbs.; 1:3:6 concrete will weigh about 154 lbs.; 1:3 mortar, 140 lbs. If the masonry contains 30 to 40 per cent. granite "plums," 55 to 65 per cent. of 1:3:6 concrete, and 5 per cent. mortar it weighs about 157 to 158 lbs; if 30 to 40 per cent. limestone "plums," about 149 to 150 lbs., depending on the specific gravities of the aggregates in the concrete and of "plums" or large stones, and on density of the masonry.

GROUT*

For grouting dam foundations, tunnels, shafts and other places, especially where cold water under pressure is encountered, a quick setting cement is often necessary. Grout sets much more slowly than mortar. Portland cements endure this severe use better than natural cements because the latter cannot stand the larger proportion of water. Natural cement grout has been found of a putty-like consistence after several months.

Table 199. Proportions for Grout in East River Tunnels, Pennsylvania R. R (New York)

Quantity of cement	Natural cement	Portland cement
Pounds per barrel, net	380.0	380 0
Cubic feet per barrel	4 5	3 5
Barrels per cubic yard, neat grout	4 6	6 1
Barrels per cubic yard 1:1 grout	2 6	3 6

(Noble and Woodward)

PUDDLE†

Puddle is a mixture of clay, sand and gravel, moistened and thoroughly compacted into certain parts of hydraulic structures to prevent seepage or percolation of water. Proportions differ with the characteristics of the ingredients and the details of use. In some places nearly, or quite satisfactory, natural mixtures are found; in some instances, mixing is done crudely in the trench, in others a pug mill or similar machinery is employed and as much care taken as with concrete. It is used as cores in earth dams, on the bottoms and slopes of reservoirs where they are pervious, beneath bottoms and on outsides of walls of filters and masonry reservoirs. Ingredients should be carefully selected and mixture determined by trial. Stones large enough to interfere with thorough mixing and compacting must be removed. Use only water enough to insure moistening throughout the mass. Clay should not be in so large proportion as to make the puddle too slimy or cause shrinkage and cracks

* For proportions used on Rondout pressure tunnel see E R, Dec 30, 1911, and page 285 For other articles on grouting Catskill aqueduct structures, see E. R., Apr. 8, 1911 and Feb. 28, 1914; E. N., Feb. 4, 1915; T.A.S.C.E., Vol. 78, 1911; and especially articles by J. F. Sanborn, E. R., Apr. 15, 1916, *et seq.*

† (See also page 199 and page 200.)

in drying. Thoroughness of compacting has much to do with prevention of cracks. Puddle should not be allowed to dry out; if work suffers long interruption, puddle should be sprinkled occasionally with water or covered with canvas, boards, moist earth or other protection. In finished structures, puddle should be protected by earth, masonry or stone paving so that it will not be eroded by water, dried out, or exposed to frost. For the Philadelphia filter plants, puddle was made of 1 part plastic red clay from Swedeland, Pa., 1 part red clay from Delaware City, Del., and 2 parts sandy gravel or broken stone smaller than $\frac{3}{4}$ in. in greatest diam. Clay was ground in pug mills to reduce lumps and thoroughly mixed with stone or gravel; water was added to make stiff paste. While plastic, it was spread in an 8-in. layer on a prepared earth bottom, in large areas, and while drying repeatedly rolled with grooved rollers weighing $\frac{1}{2}$ ton per lin. ft. of roller, until all shrinkage cracks were closed; thickness was reduced to 6 in. and two more layers added, making total thickness, 12 in. Such puddle was very impervious and strong. The concrete of the filter bottoms was placed on it.

Fanning gives the following formula for an especially dense, strong puddle, which will successfully resist water, rodents and eels:

Table 200. Gravel-clay Puddle; Proportions of Ingredients

Materials	Per cent. voids	Cu yds required	
		Theoretically	Practically
Screened coarse gravel	28–30	1 00	1 00
Fine gravel..... ..	30	0 28	0 35
Sand.. ...	33	0 08	0 15
Clay..		0 03	0 20
Total materials .		1.39	1 70
Resulting puddle		1 00	1 30

In practice, 7 measured cartloads of coarse gravel mixed with 3 of fine made about 8 loads, which were spread in 2-in. loose layers in the trench; on the gravel, 2 loads of clay were spread, and lumps broken, and on top 1 load of sand. Clay loads were slightly smaller than the others. This triple layer, thoroughly mixed with a harrow, was then moistened to a kneading consistence and thoroughly compacted into a solid mass by a 2-ton grooved or ring roller. Such puddle may cost as much as concrete (a larger bulk being used) and in most cases would not be as satisfactory.

CHAPTER XXVII

NON-FERROUS METALS, ALSO CORROSION OF IRON AND STEEL

BRONZES

Sluice-gate and Valve Seats. Bronzes *A* and *B* were used in the sluice gates of the Pathfinder and Shoshone dams, U. S. Reclamation Service (O. H. Ensign); *C* and *D* were used for Catskill aqueduct. These classes have the following approximate compositions in percentages:

Metal	Class			
	A(a)	*B*(a)	*C*(b)	*D*(b)
Copper	83 5	81 8	82 8	82 7
Lead. .. .	8 1	4 9	8 0	4 9
Zinc	4 5	5 3	4.4	5 3
Tin.....	4 9	7 1	4 8	7 1
Total.	101.0	99.1	100.0	100.0

(*a*) by analyses of test pieces; (*b*) by specification. To test bronzes *A* and *B*, pieces 1 in. wide and 2 in. long were moved backward and forward on pieces of opposite composition, 1 in. wide, with 6-in. stroke, under a load of 3200 lbs. per sq. in., at planer speeds; no squealing, chattering, nor dust developed after several hundred strokes. Catskill aqueduct specifications allowed a variation in per cent. of any one ingredient not exceeding 0.75, and required: A piece of *C* bronze, at least 16 in. long and 2 in. wide, finished on one side, shall be firmly secured, and a piece of *D* bronze, 8 in. long, 2 in. wide, similarly finished, rubbed back and forth along its surface for at least 500 strokes at speed of at least 35 ft. per min., the two metals being forced together with a pressure of not less than 1000 lbs. per sq. in., and the surfaces lubricated with water only; neither metal shall be abraded nor cut, nor shall there be any chattering nor screaming. Specimens from one melt of each bronze having satisfactorily passed this abrasion test, and chemical analysis, following melts were passed on analysis and behavior during machining. *C* and *D* bronzes have been successfully made by several manufacturers. Tests were usually made in a planer.

Aluminum-bronze for High-pressure Castings. In a series of tests in England (1907–1910) by Carpenter and Edwards on metals for high-pressure cylinders and valves the conclusion was reached that aluminum-bronze, with about 9 and 10 per cent. of aluminum, gave the best results. The moment that the fresh surface of the molten metal is exposed to air, a tough tenacious film of alumina forms on the surface, being removable by deoxidizers or a combination of fluxes. This film protects the metal beneath, but if the dross gets into the casting, as it often does, the strength is reduecd. To keep the

occluded dross down to the minimum, molten metal must be poured quietly as agitation increases the mixture of dross. Aluminum-bronze castings undergo excessive shrinkage, equal to steel. Experimenters found shrinkage troubles could be overcome by an expert molder, if sufficiently large risers and gates were used, and the casting designed free from sharp corners. Aluminum-bronze is claimed to be the most homogeneous of known bronzes; crystallization has never been found.—(E. N., Mar. 30, 1911.)

Cracking of Brass and Bronze. Pipes, tubes, bars, bolts, rods, plates and wires of common brass, manganese bronze and naval brass sometimes crack where not under external stress and for no *apparent* reason. The cause, so far as now determined, commonly is: Too severe cold drawing in finishing, improper heat treatment, or lack of annealing, or wrong methods of final fabrication. Such cracking rarely develops within 1 month of manufacture, and may not appear for several months or even years. These defects should be guarded against for important uses. In the present state of the art, the only safeguard seems to be insistence on the products of reliable manufacturers, and having all work, especially such as involves any heating or deforming of the metal, done only by parties experienced and skilled in the particular kind of work to be done. The standard physical and chemical tests now in use do not detect the concealed tendencies to the defects mentioned above. Exposure to extreme cold, as during a winter, expedites the cracking. Until the Government, some association of manufacturers, or other organization, shall have thoroughly investigated copper alloys so as to determine the right methods of manufacture and fabrication, and dependable tests for inspection, manganese bronze, naval brass, common and other brasses must be used with caution, in rolled, drawn and similar forms. Castings of these alloys, so far as known, are subject simply to the usual mishaps of the founders' art, and are very useful for many engineering purposes.*

In view of such behavior, L. D. Van Aken, Supt. National Brass & Copper Co., recommends the following percentage: copper, 60 to 63; tin, 0.5 to 1.5; zinc, remainder; iron, 0.10, maximum; lead, 0.2, maximum. The following physical requirements take cognizance of the size of the metal:

Diameter or thickness, in.	Pounds per sq. in.		Elongation	
	Tensile strength	Elastic limit	Per cent. in 8 in.	Per cent. in 2 in.
$\frac{1}{4}$–$\frac{1}{2}$	60,000	27,000	25	34
$\frac{1}{2}$–1	58,000	26,000	28	40
Over 1	54,000	25,000	28	40

E. R , Aug. 22, 1914.

Blackening Brass or Bronze. It is sometimes desirable to blacken articles of brass or bronze to avoid tarnishing, or to disguise them as protection from theft. It can be cheaply accomplished by the following methods: (1) oxidizing in a flame, as a Bunsen burner; (2) copper carbonate process

* For many years, brass founders have repaired defects in brass and bronze castings by "burning in" or other process of welding. Some recent experiences demonstrate that such repairs cannot be permitted on hollow castings to contain water under pressure. If made by a man of rare skill and long experience, the repairs may be successful, provided due regard is had for all the conditions in each case.

(see Henley's Twentieth Century Recipes), briefly described as follows: the solution consists of 1 lb. of copper carbonate and 2 gals. of strong ammonia. The articles should be cleaned, before dipping, by boiling in a strong potash solution, and then rinsed and dipped in the copper solution, which has previously been heated to from 150° to 175° F., and left in the solution until the required tint is produced.

CORROSION OF METALS

Copper oxidizes very slowly in the atmosphere, being covered with a film of carbonate, commonly called "verdigris," forming a protective coating, which, however, erodes or is slightly solvent in the weather.

Tin is unaffected by corrosion in the air, beyond the formation of a thin film on the surface.

Zinc is easily acted on by moist air; a film of oxide is soon formed, however, which protects the metal. If the moisture of the atmosphere contains acid, the zinc may be destroyed.

Lead. The chemical properties of lead are peculiar and present some very remarkable contrasts. While it resists sulphuric and hydrochloric acids in a far higher degree than iron, zinc, or tin, it is readily attacked by weak organic acids, and is slowly dissolved, even in pure water containing air. It is soon extensively corroded when exposed to moist air in the presence of carbonic acid. Lime mortar, lime putty, and lime water will attack lead; if the mortar is very alkaline, the effect will be greater. John Newman* reports an instance of lead pipe embedded in cement found deeply corroded; the action was destructive near the end of the pipe next to a basin, diminishing as the pipe receded from the water; the same authority gives several cases of damaging corrosion of lead pipe from contact with mortar or cement. A weighed sample of lead immersed 8 days in water seeping through concrete walls in a tunnel under sea water, at New York, was taken out daily, washed, dried and weighed; progressive corrosion took place, amounting to 0.132 per cent. or at the rate of 6 per cent. per year. In a bath establishment, 2 lead pipes pass through a concrete floor, a layer of asphalt, and tile embedded in cement mortar; 1 pipe for hot and the other for cold water; the room temperature, practically constant throughout the year, was 80° F. Water, high in free carbonic acid, lime and magnesia, was freely used on that floor; the lead pipe for hot water was corroded completely off in 5 to 10 yrs.; the cold water pipe was deeply corroded; no probability of electrolytic action existed. In 1860–61 a portion of St. Petersburg† was served by a system of lead mains; in 10 to 15 yrs. these pipes had become so damaged by corrosion as to necessitate extensive changes; the trouble was attributed to a local peculiarity of soil, the exact nature of which was never ascertained. In ordinary atmospheric corrosion, lead is one of the most durable of the common metals, undergoing no change in dry air or in water perfectly free from air; it is only slightly affected by hard waters or dilute solutions of either hydrochloric or sulphuric acids; but is readily dissolved by water high in nitrates and by dilute nitric acid; waters which actively corrode lead are those with a slightly acid reaction, from peaty swamps;

* "Metallic Structures: Corrosion and Fouling and their Prevention," Spon, 1896.
† Petrograd.

soft waters are particularly unsuited for conveyance in lead pipes. If exposed to clean, soft water containing the normal quantity of dissolved oxygen, the lead is oxidized to hydroxide, which dissolves; the waters which act least on lead are those which contain carbonate of lime, phosphate of lime, and, in a less degree, sulphate of lime. Newman quotes various experiments to show that lead water pipes should be kept full of water all the time to prevent deterioration. Since lead acts as a cumulative poison, its salts produce serious results if taken into the human system even in very minute quantities for a length of time.

Under whatever conditions lead withstands water or acids, bronzes are similarly unaffected, while under other common conditions in which bronzes are practically untouched, lead would be destroyed. Lead-lined cast-iron pipe has been destroyed by well-defined electrolysis, and in these cases the lead was eaten through as well as the iron (or steel). Lead affords doubtful protection to iron or steel water pipes and under some conditions might prove a disadvantage. The coefficient of expansion of lead is 2.5 that of steel, hence it is questionable whether in large pipes or specials, lead lining would remain in absolute contact. The high specific gravity of the lead, coupled with its great expansion, confers a tendency to work downward if in a vertical or sloping position. If water finds its way between the steel (or iron) and the lead lining, lead being electropositive to steel, energetic galvanic action and rapid corrosion of the pipe metal would follow.

Cast Iron rusts slowly in air or in fresh water, but is rapidly corroded in salt water (see p. 407) in which it gradually becomes soft. Mallet (Proc. Inst. C. E., Vol. 1, 1840) found that the rate of corrosion decreased with the thickness of the casting, being from 0.1 to 0.4 in. in depth during a century for castings 1 in. thick. Old cast-iron pipes taken up or cut into from time to time, are found in good condition in many localities; old pipe thus removed is often in good enough condition to be relaid. Gas companies in New York have had practically no trouble from the corrosion of cast-iron pipe except to a limited extent along the river front. All mains are now of cast iron. Experience with wrought iron, and more particularly with steel, pipe has been unsatisfactory on account of corrosion. At Rochester, N. Y., no rust leaks had occurred in a cast-iron conduit 14½ miles long after 32 yrs.' service. During the same period in a wrought-iron conduit 13 miles long, there had been seven rust leaks, whereas in a parallel steel conduit 29 miles long, given the best protective coatings then known, passing through the same kind of soil and thus subjected practically to the same conditions, there were 164 rust leaks in 16 yrs., or one-half the period of service of the cast-iron conduit.

Experience seems to show that cast-iron resists corrosion better than either wrought-iron or steel. Cavallier, director of foundries at Pont-a-Mouson, France, mentions an intake pipe at Paris, extending into the river Seine, laid in 1802. Taken up after more than a century of immersion, it was found to be in fine condition. The same authority reports that the fountains at Versailles are supplied through cast-iron pipes laid in 1665. (Jl. N. E. W. W. Assn., June, 1904.) For external corrosion, see Trans. Am. Soc. C. E., Vol. 78, 1915, p. 806.

Experiments at Louisville (Report on Water Supply of St. Louis, 1902, p. 35) demonstrate the fact that the removal of suspended matter by filtration, without the use of coagulants, accelerates corrosion 100 per cent., showing that river silt acts as a preservative of iron when the water contains free carbonic acid and oxygen in solution. It is supposed that this preservative action is effected by mixing of suspended particles with ferric hydrate of iron, thus shielding the surface of the metal where corrosion is taking place.

Iron structures immersed partly in salt and partly in fresh or brackish water corrode somewhat rapidly on account of the galvanic action between portions of the iron immersed in liquids of different densities. (Jl. Assn. Eng. Soc., Vol. 4, 1885). At St. Johns, New Brunswick, a 4-in. main, in use 33 yrs., failed in marsh mud under a pressure of 60 to 65 lbs. per sq. in. The outside of the pipe had undergone a softening process at the break, which was along some air cells in the body of the shell. The film of metal covering the cells and sand-holes was about $\frac{1}{8}$ in. thick, and could be cut easily with an ordinary knife. Below the films and air cells, the metal seemed as clean as when first laid.—(Eng. News, Jan. 4, 1894.)

Bronzes are not affected by ordinary conditions of earth or atmosphere. Evidence is afforded by numerous objects collected among the ruins of ancient cities in various parts of the world, many having inscriptions still legible, although they have been exposed to air and moisture for more than 20 centuries. A great quantity of prehistoric bronze axes, knives, etc., in good preservation, have been taken from lakes in southeastern France. Modern bronzes, excepting brass and Muntz metal which dissolve in sea water, have apparently the same freedom from corrosion under ordinary conditions. Manganese, Tobin, and phosphor bronzes are little affected by sea water and have proved satisfactory where brass and steel have proved useless. Bronze in the gates of the Old Croton aqueduct, in service since 1842, so far has been unaffected. No trouble has been experienced from corrosion of bronze in stopcocks or hydrants in Croton Waterworks, New York City.

Alloys and Steel in Moist Earth. In 1908 and 1909, at the Laboratory of the Catskill aqueduct, New York Board of Water Supply, there were embedded in rich earth, kept moist with Croton water and the occasional addition of 0.5 per cent. solutions of chlorides of sodium and magnesium, 6 weighed rods, 5.98 in. long, 0.47 in. in diam.; after 6 months they were washed, dried and

Table 201. Loss of Weight in Grams of Metals in Moist Earth

Sample	Original weight	Loss, first 6 months		Loss, second 6 months	
		Total	Per sq in	Total	Per sq in.
Phosphor bronze.	171 87	0 16	0 0173	0 06	0 0065
Tobin bronze.	162 11	0 19	0 0206	0 07	0 0076
Monel metal..	160 76	0 19	0 0206	0 08	0 0087
Parson's manganese bronze.	161 65	0.19	0 0206	0 08	0 0087
Muntz metal... .	163 85	0 55	0 0596	0 10	0 0108
Steel. .	139 58	1 45	0.1571	1 46	0.1581

reweighed. All showed more or less oxidation, Monel metal presenting the least change in appearance. They were reembedded for 6 months in the same earth, and kept moist by the addition of Croton water alone (table 201).

Alloys Embedded in Concrete and Submerged. The specimens were partially embedded in concrete blocks, and immersed in Esopus Creek, Catskill Mts., N. Y., May 18, 1908, and removed Aug. 4, 1910. The most corrodible specimen lost at an average rate of 114 millionths in. per year, based on the average loss over the whole surface of each specimen, determined by careful weighing before and after the test. At this rate it would require 1100 yrs. to corrode off ⅛ in.; it would take 4600 yrs. to corrode off ⅛ in. of the least corrodible. Specimens embedded in the richer concrete were corroded more than those in the leaner concrete. This suggests that the cement accelerated the corrosion. Corrosion of the embedded portions may have been 50 per cent. greater than stated, and that of the exposed portion correspondingly less. The chemical compositions of these alloys are as follows:

Table 202. Chemical Compositions of Copper Alloys. Tested for Corrosion in Catskill Aqueduct Laboratory

Name of alloy	Copper	Zinc	Tin	Lead	Nickel	Iron	Manganese	Phosphorus	Specific gravity
Muntz metal ..	59 16	40 36	......	0 41	......	0 07	......	......	8.41
Parson's manganese bronze.	58.92	39.48	0.90	0.12	......	0.57	0 01	......	8.41
Tobin bronze ...	60 65	38.71	0 49	0 49	......	0 06	. ....	......	8.43
Phosphor bronze.	93 77	.....	6 04	trace	......	0 02	......	0.17	8 93
Monel metal.	26.40	1 59	......	......	69.55	2.46	......	.....	8 83

Round rods, ½ in. by 6 in., placed in 6-in. concrete cubes two-thirds their lengths, Apr. 1, 1908. Concrete mix: 1:1½:3 and 1:3:6, with a specimen of each metal embedded in a cube of each mix. Cement: Giant Portland with the following chemical analysis: SiO_2, 23.74 per cent.; Fe_2O_3, 1.86; Al_2O_3, 5.04; CaO, 61.83; MgO, 3.63; SO_3, 1.44; alkalies, water, CO_2, etc., 2.46. Sand was from Cow Bay, Long Island, passing No. 4 sieve. Gravel was from the same place, running from 1½ in. to No. 4 sieve in size. The following is an analysis (expressed in parts per million) of Esopus Creek water, taken 0.5 ft. below the surface, on Sept. 7, 1908: Nitrogen as nitrites, 0.002; as free ammonia, 0.004; as albuminoid ammonia, 0.016; as nitrates, 0.1; total solids, 51; loss on ignition, 10; iron, 0.1; chlorine, 1.10; alkalinity, 18.50; hardness, 27.30. Color test gave 10; turbidity, plus, and odor very faint, earthy.

Table 203. Loss by Corrosion of Copper Alloys in Concrete (in 2 Yrs. and 2 Months)

Alloy	Concrete mix, 1.1½:3		Concrete mix, 1:3:6	
	Weight, grams	Thickness, millionths of an inch	Weight, grams	Thickness, millionths of an inch
Muntz metal..... . ..	0.25	214	0 29	248
Parson's manganese bronze.	0.23	179	0.19	163
Tobin bronze........ ...	0.21	180	0 14	122
Phosphor bronze..... ..	0.12	97	0 08	65
Monel metal....... ...	0 07	59	0 04	33

When the rods were taken out of the concrete a few days after removal from the creek, adhesion was good, particles of mortar adhering in all cases except that of Monel metal. There were considerable deposits of verdigris (probably copper carbonate) on all rods except Monel, but the staining did not penetrate into the concrete to any extent. In all cases the verdigris was most abundant at the end of the embedded portion. The exposed portions, again excepting Monel metal, were covered by a thin dark coating amounting to nothing but tarnishing. Monel metal was bright on both the embedded and exposed portions. No pitting or other evidence of localized corrosion was observable. Muntz metal, closely resembling ordinary brass, was the most corrodible.

Corrosion of Steel Pipe. Electrolysis occurs as a natural process in buried iron or steel pipes inadequately protected, if the ground conditions afford media for solution and electrolytic conduction (see also p. 416). The presence of minute amounts of soluble salts in wet soils may lead to destructive electrochemical action. In the Rochester pipe line, soil conditions played a leading part in corrosion. The damage was confined to very wet soils. A portion of the Portland, Ore., steel pipe laid in trench entirely in sand in the river bottom was found in excellent condition; in wet clay the pipe was much pitted and rusted. The Atlantic City 30-in. steel main illustrates the fact that earth and water of salt marshes are actively corrosive to buried metallic structures; the soil was very acidulous. When clay soils are impregnated with salt or brackish water, they are much more corrosive than sandy soils similarly situated. At New Bedford, Mass., a steel pipe passes through 2 miles of peaty swamp. Gravel was hauled in to use as pipe covering; there has been no trouble from corrosion. Peat is considered non-corrosive by some authorities. Peat may be corrosive or non-corrosive, depending upon its acidity or alkalinity. At Cambridge, Mass., the greatest damage was done where the soil was a moist sandy clay. Experience seems to show that both wrought iron and cast iron resist corrosion better than does steel.

Preventing Corrosion of Iron and Steel. *Mortar and Concrete.* Concrete is not an electric insulator; a small fraction of an ampere of electricity will corrode steel embedded in concrete and disintegrate the concrete. In sea water concrete offers less resistance than in fresh. (A. A. Knudson, Trans. Am. Inst. E. E., Vol. 26, 1907). Iron embedded in fresh mortar rusts rapidly;* in pure cement it remains unrusted, even when kept under water. (L. E. Andes, "Iron Corrosion," p. 26.) A piece of iron in a sandstone masonry pedestal about 200 yrs. old was badly rusted. A piece of cast iron from the Tower Subway shell, London, cut out after 16 yrs. was in good condition. It had been coated with coal tar, with an outer coat of hydrated lime. (James Forgie, Trans. Am. Soc. C. E., Vol. 50, 1903.) In a steel vessel, after 10 yrs.' use without care, it was found that the unprotected floor in the boiler space was completely gone; under the boilers, where there had been a coating of practically pure Portland cement, the shell and angle bars were found in absolutely perfect condition. (John Ried, Trans. Inst. Engrs. and Shipbuilders of Scotland, Vol. 45, 1901.) A standpipe and tower at Louisville, after 21

* Not borne out by other observations.

yrs. were in a badly rusted condition; they were cleaned and coated on the inside with 1:1 Portland cement mortar. The cleaning and coating were done in 8 hrs., water being pumped the other 16. After some trouble, pipe was finally coated in a satisfactory manner. (Report, Louisville Water Co., 1883, p. 22.) C. Herman says that this standpipe blew down in March, 1890; cement lining was in perfect condition and rust action completely stopped. Cement paint is largely used by railways in France on bridges, the iron being brushed, dampened and given 2 coats of cement and sand. Dr. Goslich states that iron spirit tanks, painted inside with Portland cement, are universally employed in European distilleries. (S. B. Newberry, Assoc. Expanded Metal Soc., Chicago, Ill., Feb. 20, 1902.) L. L. Buck argues that limestone aggregate should not be used in concrete which is to come into contact with steel, as a soluble stone in contact with the steel may be reached by the water, and a cavity ⅛ to ¼ in. deep formed.—(Trans. Am. Soc. C. E., Vol. 30, 1898, p. 31.)

Painting Steel Work. A good paint for the first coat consists of 20 lbs. of red lead and 1 lb. of lampblack in about 3 qts. boiled linseed oil, the red lead and lampblack being ground in. This will suffice to cover 50 sq. yd. For the other coats some durable oil or asphaltum paint should be used. For painting iron or steel work a good formula is: 1 coat red lead in oil, 1 coat elastic paint, 1 coat or more of desired color containing zinc oxide (40 per cent.), lead oxide (50 per cent.), and asbestos filler (10 per cent.). When repainting, all metal surfaces should first be carefully cleaned by a sand blast or by steel brushes and scrapers (good scrapers can be made by drawing the temper from old files, and sharpening like chisels). Rust spots should be removed even if considerable time be required, for if such spots are covered by paint, corrosion will still continue underneath and as the rust swells it will blister and push off the paint. Paint will harden better if applied when the metal is warm, and in any event the metal should be thoroughly dry. A steam hose is useful for drying steel work.

Flange Joints: Gaskets. For bronze flanges under great pressure where durability was essential, soft lead wire gaskets were adopted on Catskill aqueduct. Table 203a gives results of some tests. "Enduro" and "Velumoid" paper gaskets have also been used successfully.

Table 203a. Lead and Copper Wire Gaskets: Compression Tests

Material	Initial diam in.	Length, in. Initial	Length, in. Final	Load, lbs Initial	Load, lbs Final	Final thickness, in.	Remarks	
Lead	0 387	5 0	......	2,310	10,150	0 250	Down flat	Each load (5 to 8 loads) on 1 min.
Lead	0 500	5 0	......	2,970	26,600	0 253	Down flat	
Lead	0 382	5 0	......	2,300	10,000	0 253	Down flat	
Lead	0 383	2 47	2.52	1,000	14,000	0 253	Down flat	
Lead	0.499	2 50	2.67	1,500	12,000	0.252	Down flat	
Annealed Copper	0 311	5 01	.....	3,490	70,000	0 253		
Annealed Copper	0.311	5 0		8,000	64,000	0 250		

Wrought-iron bolts and iron work in foundations of Third Ave. Bridge, New York, put in before 1860 were taken out in 1894 in perfect condition, grease being still on the threads, continuously submerged in salt water. Nuts were easily started with wrench (J. B. Goldsborough).

CHAPTER XXVIII

CAPACITY AND CONVERSION TABLES

For more complete tables, see Hering's "Conversion Tables" (John Wiley Sons, Inc) and Trautwine's Engineers' Pocketbook (The Trautwine Co.)

QUANTITY OF WATER

Gallon, New York Statute measure (1829) = 8 lbs. of distilled water or 221.184 cu. in., with barometer at 30 in., and temperature, 39.83° F. The legal gallon of United States is the old British wine gallon, containing 231 cu. in., being the contents of a cylinder 7 in. diam. and 6 in. high; it contains 8.3389 lbs. of water at maximum density, weighed in air at 30 in. mercury pressure, and 62° F. Also = 0.1337 cu. ft. = 0.8327 imperial gal.

Imperial gallon contains 10 imperial lbs. of distilled water at 62° F. weighed in air of the same temperature, and with barometer at 30 in. 1 imperial gal. = 1.200 U. S. gals. = 277.27 cu. in. = 0.160 cu. ft.

1 ton of water (2000 lbs.) = 240 gals. ±.

1 drop water, 4° C. or 39° F. = 1 ♏ (minim) (U. S.) = 0.0616 c.c. = 0.00376 cu. in. = 0.0000162 gal. Experiments by E. G. Bradbury* indicated that 16,000 drops of water from metallic surfaces are approximately equal to 1 gal., a somewhat larger volume per drop than given by Kuichling who found 1 drop per sec. equal to 3 gals. per day.

1 million gals. = 3.07 acre-ft.

1 million cu. ft. = 22.95 acre-ft.

1 million gals. daily (mgd.) = 1.547 cu. ft. per sec. = 694.44 gals. per min.

1 drop (♏) or 1 minim, per sec. = 1.40 gals. per day.

1 sec.-ft. or cu. ft. per sec. (cfs.) = 448.8 gals. per min. =1 in. (roughly) per acre per hr. = 646,272 gals. per 24 hrs. = 38.4 Colorado miner's in. = 50 Utah or Nevada in. = 40 California (Law of Mar. 23, 1901) or Arizona in. = 1.98 acre-ft. in 24 hrs. = 59.5 acre-ft. in 30 days = 724.5 acre-ft. in 1 yr. = 1 acre-ft. in 12 hrs. 6 min. = 86,400 cu. ft. in 24 hrs. = 0.646317 mgd.

1 sec.-ft. for 1 yr. = depth of 13.572 in. on 1 sq. mi.

1 in. of rainfall per month (30 days) = 0.8962 cfs. per sq. mi. = 0.5793 mgd. per sq. mi.

1 cu. ft. per sec. per sq. mi. = 1.115 in. of rainfall per month.

1 mgd. per sq. mi. = 1.726 in. rainfall in 30 days = 1.55 sec.-ft. per sq. mi.

1 mgd. per acre = 0.000000247 sec.-ft. per sq. in. = 990.22 sec. ft. per sq. mi.

1 in. on 1 acre = 27,154.3 gals. = 3630 cu. ft. = 0.0833 acre-ft.

1 acre-ft. = 43,560 cu. ft. = 325,850 gals.

1 in. on 1 sq. mi. = 17.379 million gals. = 2,323,200 cu. ft. = 53.33 acre-ft. = 0.0737 sec.-ft. per year.

*Ohio Engg., Soc., Jan., 1912.

1 in. per hr. on 1 acre = 0.652 mgd. = 1.0086 cfs. = 2.0 acre-ft. per day.

1 in. per hr. on 1 sq. mi. = 417.15 mgd. = 645.333 cfs. = 1280 acre-ft. per day.

1 in. per month on 1 sq. mi. = 0.896 cfs. per sq. mi. = 0.6 mgd. per sq. mi.

1 in. per sq. mi. per day = 26.8889 cfs. (nearly 27).

Miner's inch is defined as the quantity of water that will pass through an orifice 1 sq. in. in cross-section under a head given in various localities as 4 to $6\frac{1}{2}$ in. to the center of the orifice. In states where the miner's inch is defined by statute, it varies from $\frac{1}{40}$ to $\frac{1}{50}$ cu. ft. per sec. It is going out of use, being superseded by cu. ft. per sec. and gals. per min.

Hogsheads, Hurst's rule. Capacity in U.S. gals. = $0.0017l(P+m)^2$ where P = product of inside diam. at the heads, inches; for equal diam., d, $P = d^2$. m = inside diam., inches, at the bung. l = length, inches, between insides of heads.

Table 204. Converting Discharge in Second-feet per Square Mile into Run-off in Depth in Inches over the Area*

Discharge in sec.-ft. per sq. mi.	Run-off in inches				
	1 day	28 days	29 days	30 days	31 days
1	0.03719	1 041	1.079	1 116	1.153
2	0.07438	2.083	2.157	2 231	2.306
3	0.11157	3.124	3.236	3.347	3.459
4	0.14876	4.165	4.314	4.463	4.612
5	0.18595	5.207	5.393	5 578	5.764
6	0 22314	6.248	6.471	6 694	6.917
7	0 26033	7.289	7.550	7 809	8.070
8	0.29752	8.330	8.628	8 926	9.223
9	0.33471	9 372	9.707	10.041	10.376

Note. For partial month, multiply the values for 1 day by the number of days.

Table 205. Converting Discharge in Second-feet into Run-off in Acre-feet*

Discharge in sec.-ft.	Run-off in acre-ft.				
	1 day	28 days	29 days	30 days	31 days
1	1.983	55 54	57 52	59.50	61.49
2	3 967	111.1	115 0	119.0	123.0
3	5.950	166.6	172.6	178.5	184.5
4	7.934	222.1	230.1	238.0	246.0
5	9.917	277.7	287.6	297.5	307.4
6	11.90	333.2	345.2	357.0	368.9
7	13.88	388.8	402.6	416.5	430.4
8	15 87	444.3	460.2	476.0	491.9
9	17.85	499.8	517.7	535.5	553.4

Note. For partial month, multiply values for 1 day by the number of days.

VELOCITY

1 ft. per sec. = 0.682 mi. per hr.

1 mi. per hr. = 1.467 ft. per sec. = 88 ft. per min.

* Extracts from Water-Supply Paper 324, Surface Water Supply of the United States—1912, Part IV. St. Lawrence River Basin; by C. C. Covert, A. H. Horton and W. G. Hoyt

POWER

1 hp. requires 8.80 sec.-ft. falling 1 ft. = 5.68 mgd. falling 1 ft.*
1 mgd. falling 1 ft. = 0.176 hp.*
1⅓ hp. = about 1 kw.
1 cu. ft. per sec. falling 1 ft. = 0.114 hp.

To calculate water power quickly: $\frac{\text{sec.-ft.} \times \text{fall in ft.}}{11}$ = net horsepower on waterwheel realizing 80 per cent. of the theoretical power.

PRESSURE

1 ft. head or depth of water = 0.433 lb. per sq. in. = 0.029 atmosphere.
1 lb. per sq. in. = 2.307 ft. of water = 0.068 atmosphere.
1 atmosphere = 33.90 ft. of water = 14.7 lbs. per sq. in.
1 ft. water column = 0.883 in. mercury.
1 in. mercury column = 13.59 in. water.

Table 206. Converting Inches Vacuum into Feet Suction

In. vacuum	Ft. suction	In. vacuum	Ft. suction	In. vacuum	Ft. suction	In. vacuum	Ft. suction
¼	0 28	8¼	9.35	16¼	18 42	24¼	27 50
½	0.56	8½	9 64	½	18 71	½	27 78
¾	0.85	8¾	9 92	¾	18 99	¾	28.07
1	1.13	9	10.21	17	19.28	25	28 35
1¼	1.41	¼	10.49	¼	19 56	¼	28.63
1½	1.70	½	10 77	½	19 84	½	28 91
1¾	1 98	¾	11.06	¾	20.13	¾	29.20
2	2.27	10	11.34	18	20.41	26	29 48
2¼	2.55	¼	11 62	¼	20.70	¼	29.76
2½	2 84	½	11 90	½	20 98	½	30 05
2¾	3 12	¾	12 19	¾	21 27	¾	30 33
3	3.41	11	12.47	19	21.55	27	30.62
3¼	3.69	¼	12.75	¼	21 83	¼	30.90
3½	3.98	½	13.04	½	22 11	½	31.19
3¾	4.26	¾	13 32	¾	22 40	¾	31.47
4	4.54	12	13.61	20	22.68	28	31.75
4¼	4.82	¼	13.89	¼	22.96	¼	32.03
4½	5 11	½	14 18	½	23.24	½	32 32
4¾	5.39	¾	14.46	¾	23 53	¾	32 60
5	5.67	13	14.74	21	23.81	29	32.89
5¼	5.95	¼	15.02	¼	24.09	¼	33.17
5½	6.23	½	15.31	½	24.38	½	33 46
5¾	6.52	¾	15.59	¾	24.66	¾	33.74
6	6.80	14	15 88	22	24.95	....	....
6¼	7.08	¼	16.16	¼	25.23	....	....
6½	7.37	½	16.45	½	25 51	....	....
6¾	7.65	¾	16.73	¾	25.80	....	....
7	7.94	15	17.01	23	26.08	....	....
7¼	8.22	¼	17.29	¼	26.36	....	....
7½	8.50	½	17.57	½	26.65	....	....
7¾	8.79	¾	17.86	¾	26.93	....	....
8	9.07	16	18.14	24	27.22	....	....

Table 207. Pressure Equivalents

Lb. per sq. in.	Lb. per sq ft.	Lb. per circular in	Kilogramme per sq centimeter	1000 dynes per sq centimeter	In of mercury	Millimeters of mercury, 32° F.	Water pressure, feet, head	Water pressure, meters, head	Atmosphere
1 0	144 0	0 78540	0 0703	69 0	2 0376	51 63	2 307	0 703	0 06793
0 00694	1 0	0 00545	0 000488	0 00479	0 01415	0 3585	0 01602	0 00488	0 000472
1 273	183 3	1 0	0 08952	87.845	2 594	0 6573	2 937	0 8954	0 08648
14 223	2048 0	11 17	1 0	981 3	28.98	734 2	32 81	10 0	0 966
0 01449	2 086	0 01138	0 00101	1 0	0 02952	7 48	0 03343	0 01013	0 00984
0 4912	70 731	0 38579	0 03453	33 880	1 0	25 35	1 1334	0 3454	0 03336
0 01937	2 789	0 01521	0 001362	1 336	0 03947	1 0	0 04468	0 01362	0 001316
0 4335	62 425	0 34128	0 03048	29 91	0 882	22 38	1 0	0 30491	0 029448
1 422	204 76	1 1169	0 1	98 13	2 8975	73 42	3 281	1 0	0 0966
14 72	2119 68	11 562	1 035	1015.8	29 92	760 0	33 96	10 352	1.0

Table 208. Conversion Factors for Units of Pressure*

Unit	Ft. of water	Log	In. mercury	Log	Lbs. per sq in.	Log	Lbs. per sq ft	Log
Lb. per sq in . .	2 308	0 3632	2 037	0 3090	1 0	0.0	144 0	2 1584
Lb. per sq ft	0 01603	8 2048–10	0 01414	8 1506–10	0 00694	7.8416–10	1.0	0.0
In. of mercury . .	1.133	0 0542	1 0	0 0	0 4910	9 6910–10	70 699	1 8494
Ft. of fresh water	1 0	0.0	0 8826	9 9458–10	0 4333	9 6368–10	62.4	1.7952
Ft. of sea water	1 025	0 0107	0 9047	9 9565–10	0 4442	9.6475–10	64 0	1 8062
Atmosphere	33 923	1 5305	29 942	1.4763	14 70	1 1673	2116 8	3 3257

Specific gravities used above; distilled water, 1.000; sea water, 1 025; mercury, 13 5956.

* Hughes and Safford, Hydraulics, 1911, p. 6.

Table 209. Altitudes, Barometer and Wind*

State, country or territory	Place	Altitude, ft. above sea level	Barometer, inches of mercury			Velocity of wind, miles per hr.	
			Annual mean	Aver. annual max	Aver. annual min	Annual mean (20 yrs)	Max recorded 1909–14
Alabama... . . .	Birmingham	700	29.32	29 77	28 63	10	48
Alaska......	Sitka	90	29.79	29 99	29 48	---	---
Arkansas . .	Little Rock	357	29 66	29 80	29 55	6	59
Arizona ..	Phoenix	1108	28.75	28.89	28 62	---	40
Arizona	Yuma	141	29 73	29.92	29 58	---	44
California......	Los Angeles	338	29 62	29.73	29 52	4	42
California .	San Francisco	155	29 86	29.99	29 75	10	64
Colorado	Denver	5291	24 71	24.82	24 58	7	75
Connecticut	New Haven	106	29 91	30.04	29 79	---	58
Cuba	Havana	57	29.92	30.07	29 82	---	---
District of Columbia	Washington	112	29 94	30.01	29 82	5	68
Florida	Jacksonville	43	30 01	30.13	29 91	8	75
Georgia ..	Atlanta	1174	28 83	28 93	28 74	10	66
Georgia	Savannah	65	29 99	30.11	29 89	7	88
Idaho........	Boise City	2739	27 17	27.33	27 06	4	55
Illinois.	Cairo	356	29 67	29 80	29 52	8	54
Illinois.. .	Chicago	823	29 13	29.23	29 03	17	84
Illinois.	Springfield	644	29 35	29.47	29 25	9	48
Indiana.... .	Indianapolis	822	29 16	29.27	29 06	10	60
Iowa......	Davenport	606	29 37	29 49	29 26	---	48
Iowa..... .	Dubuque	698	29 27	29.38	29 16	6	60
Kansas.... .	Dodge City	2509	27 38	27.48	27 27	11	75
Kansas .	Topeka	983	---	---	---	---	58
Louisiana .	New Orleans	51	29.98	30 10	29 87	8	66
Maine	Portland	103	29 87	30 00	29 75	5	61
Maryland	Baltimore	123	29 92	30.05	29 80	---	70
Massachusetts	Boston	125	29 88	30 00	29 75	11	72
Michigan..	Detroit	730	29 23	29 33	29 13	11	86
Michigan .	Sault St. Marie	614	29 31	29.41	29 24	8	52
Minnesota . .	Duluth	1133†	29 23	29.33	29 13	14	78
Minnesota . .	St. Paul	837	29 09	29.20	28 98	8	102
Missouri	St. Louis	567	29 43	29.56	29 33	11	80
Nebraska .	North Platte	2821	27 06	27.15	26 96	10	---
Nebraska	Omaha	1105	28 84	28.97	28 73	9	66
New Mexico	Santa Fe	7013	23 25	23.37	23 12	7	53
New York .	Albany	85	29 92	30.05	29 80	6	70
New York .	Buffalo	767	29 18	29 28	29 08	11	92
New York .	New York	314	29 69	29.82	29 57	9	96
Ohio......... .	Cincinnati	628	29 38	29.49	29 28	7	59
Ohio..........	Cleveland	762	29.21	29.30	29 11	11	73
Oklahoma.... ..	Oklahoma City	1214	28.72	28.85	28 60	11	72
Oregon........	Portland	154	29 90	30.03	29.78	6	39
Pennsylvania..	Harrisburg	374	29 65	29.77	29 53	7	46
Pennsylvania.......	Philadelphia	117	29 93	30.06	29 81	10	75
Pennsylvania.. .	Pittsburg	842	29 14	29 24	29 04	7	69

* Compiled from U. S. W. B. records. † Instrument previously at 702.

Table 209. Altitudes, Barometer and Wind—(Continued)

State, country or territory	Place	* Altitude, ft. above sea level	Barometer, inches of mercury			Velocity of wind, miles per hr.	
			Annual mean	Aver. annual max.	Aver. annual min.	Annual mean (20 yrs)	Max.† recorded 1909–14
Porto Rico	San Juan	82	29.90	29.97	29 81	– – –	72
South Carolina	Charleston	48	30.02	30.14	29.91	10	94
South Dakota	Pierre	1572	28.30	28.42	28.20	9	65
Tennessee	Chattanooga	762	29.26	29.38	29.16	6	55
Tennessee	Knoxville	1004	29.02	29.13	28.92	6	42
Tennessee	Memphis	397	29.63	29.76	29.51	8	72
Texas	Galveston	54	29.97	30.11	29.86	10	84
Utah	Salt Lake City	4366	25.62	25.74	25.51	6	66
Vermont	Burlington	268	29.71	29.85	29.55	– – –	66
Virginia	Richmond	144	29.91	30.04	29.78	8	61
Washington	Seattle	123	29.92	30.03	29.79	6	64
Washington	Spokane	1943	27.96	28.11	27.86	4	52

* Hight of observing instruments.
† Compiled by searching U. S. Weather Bureau records for these years; records give for each month the max. velocity for that month.

Table 210. Equivalent Measures and Weights of Water at 4° C. (39.2° F.)

No.	U. S gals.	Imperial gals.	Liters	Cubic meters
1	1 0	0 83321	3 7853	0 0037853
2	1 20017	1 0	4 54303	0 004543
3	0 264179	0 22012	1.0	0.001
4	264 179	220 117	1,000.0	1 0
5	0 119888	0 099892	0.453813	0 0004538
6	7.48055	6 23287	28 3161	0 0283161
7	0.004329	0 003607	0 0163866	0 0000164
8	0 0408	0.034	0.1544306	0 0001544
9	325,851	271,499	1,233,444	1,233 4438

No	Lbs.	Cu. ft.	Cu. in.	Circular in. 1 ft. long	Acre-ft.
1	8 34112	0 13368	231 0	24 5096	0.00000307
2	10 0108	0.160439	277.274	29.4116	0 0000037
3	2 20355	0.035316	61.0254	6.4754	0 0000008
4	2,203.55	35 31563	61,025.4	6,475.44	0.000810
5	1.0	0 0160266	27.694	2.9411	0.00000037
6	62.3961	1.0	1,728.0	183.346	0.0000229
7	0.0361089	0.0005787	1.0	0 10613	0.00000001
8	0 340008	0.005454	9.4224	1.0	0.00000012
9	2,717,962	43,560	75,271,581	7,986,478	1.0

"Notes on Hydrology," by D. W. Mead.
1 cu. ft. of ice weighs 57.2 to 57.5 lbs. 1 cu. ft. of salt water weighs 63 9 to 64.1 lbs.

Table 211.* Horsepower of Water (per cu. ft. per sec.)

Weight of Water taken at 62.5 lbs. per cu. ft.

Head, ft.	Horsepower: 85 per cent. efficiency	Horsepower: 100 per cent. efficiency	Head, ft.	Horsepower: 85 per cent. efficiency	Horsepower: 100 per cent. efficiency	Head, ft.	Horsepower: 85 per cent. efficiency	Horsepower: 100 per cent. efficiency
1	0 096591	0 113636	190	18 352273	21 590909	370	35 738637	42 045455
20	1 931818	2.272727	200	19 318182	22 727272	380	36.704546	43.181818
30	2 897727	3.409091	210	20 284091	23 863636	390	37 670455	44.318182
40	3 863636	4 545455	220	21 25	25 000000	410†	39 602273	46 590909
50	4 829545	5.681818	230	22 215909	26 136364	420	40 568182	47.727273
60	5.795455	6 818182	240	23 181818	27 272728	430	41 534091	48 863636
70	6 761364	7.954545	250	24.147727	28 409091	440	42.500000	50 000000
80	7 727273	9.090909	260	25 113637	29.545455	450	43 465909	51.136364
90	8 693182	10.227273	270	26 079546	30 681818	460	44 431818	52 272727
100	9 659091	11 363636	280	27.045455	31 818182	470	45 397728	53 409091
110	10 625	12 500000	290	28 011364	32 954545	480	46 363637	54 545455
120	11 590909	13 636363	300	28 977273	34.090909	490	47 329546	55.681818
130	12 556818	14 772727	310	29.943182	35.227273	520†	50.227273	59 090909
140	13 522727	15 909091	320	30.909091	36 363636	540	52.159091	61 363637
150	14.488636	17 045454	330	31.875000	37.500000	560	54.090909	63 636364
160	15 454546	18 181818	340	32 840909	38 636364	580	56 022727	65 909091
170	16.420455	19 318182	350	33 806818	39 772727	650	62 784091	73 863636
180	17 386364	20 454545	360	34 772727	40 909091	750†	72 443182	85 227273

* Horsepower = (discharge per sec in lbs × head, ft ÷ 550) × efficiency factor.
Example: A stream flow of 1000 cfs., and 100 ft. drop provides 9.659 × 1000 = 9659 hp. at 85 per cent. efficiency
One cu. ft. per min falling one ft = 0 00189 hp.
By moving decimal point in second and third columns of each group, hp. for any head likely to be desired can be obtained nearly enough; closer results for some heads (*e.g.*, three or more significant figures, may be obtained by adding.

† Values for 400, 500, 600, 700, 800, etc., can be gotten from table by multiplying values for 40, 50, 60, 70, 80 respectively by 10.

Water horsepower, or useful work done in pumping, equals capacity in gal. per min. multiplied by total head in ft., and divided by 3960: For sea water, add 2.6 per cent. to results.

Table 212. Equivalent Quantities of Water

Delivered in 24 hrs., in 1 hr., in 1 min., and in 1 sec.

Gals., 24 hrs.	Gals, 1 hr.	Gals., 1 min.	Cu. ft., per sec.	Gals., 24 hrs	Gals., 1 hr	Gals., 1 min.	Cu. ft. per sec.
2,500,000	104,166 6	1,736 0	3 8681	144,000	6,000 0	100 0	0 2228
2,000,000	83,333 3	1,388 0	3 0945	129,600	5,400 0	90 0	0 2005
1,500,000	62,500 0	1,041 7	2 3205	115,200	4,800 0	80 0	0 1782
1,000,000	41,666 6	694 4	1 5472	100,800	4,200 0	70 0	0 1559
950,000	39,583 3	659 7	1.4697	100,000	4,166 6	69 4	0 1547
900,000	37,500 0	625 0	1 3925	86,400	3,600 0	60 0	0 1337
850,000	35,416.6	590 2	1 3152	75,000	3,125 0	52 9	0 1160
800,000	33,333.3	555 5	1 2378	72,000	3,000 0	50.0	0.1114
750,000	31,250.0	520.8	1 1604	60,000	2,500 0	41 6	0 0928
700,000	29,166 6	486.1	1 0830	57,600	2,400 0	40 0	0 0811
650,000	27,083.3	451.3	1 0057	50,000	2,083 0	34.7	0 0774
600,000	25,000.0	416.7	0.9283	43,200	1,800 0	30 0	0 0668
550,000	22,916.6	381.9	0.8510	28,800	1,200 0	20.0	0 0446
500,000	20,833.3	347.2	0.7736	25,000	1,041.6	17.3	0 0387
450,000	18,750.0	312.5	0.6961	20,000	833.3	13.8	0 0309
400,000	16,666.6	277.7	0.6189	15,000	625.0	10.4	0 0232
350,000	14,583.3	243.0	0.5415	14,400	600 0	10.0	0 0223
300,000	12,500 0	208.3	0.4642	10,000	416.6	6.9	0 0155
250,000	10,416.7	173.6	0.3868	7,200	300 0	5 0	0 0111
200,000	8,333.3	138.8	0 3094	5,000	208.3	3 4	0 0077
150,000	6,250.0	104.1	0.2321	1,440	60.0	1.0	0 0022

Table 213. Cylindrical Tanks*—Capacity for Depth or Length of 1 ft. and 1 in. Cu. ft., Gals., and Lbs.

Depth	1 Ft			1 In.			Depth	1 Ft			1 In			Depth	1 Ft			1 In		
Diam	Cu. ft. †	Gals.	Lbs	Cu. ft	Gals.	Lbs	Diam	Cu. ft †	Gals.	Lbs	Cu. ft	Gals	Lbs	Diam	Cu. ft †	Gals	Lbs	Cu. ft	Gals.	Lbs.
2′–0″	3.1	24	194	0.262	1.97	16	5′–0″	19.6	147	1,227	1.6	12	102	8′–0″	50.3	376	3,141	4.2	31	262
1″	3.4	26	212	0.284	2.13	18	1″	20.3	152	1,268	1.7	13	106	1″	51.3	384	3,208	4.3	32	267
2″	3.7	28	231	0.307	2.31	19	2″	21.0	157	1,311	1.7	13	109	2′	52.4	392	3,274	4.4	33	273
3″	4.0	30	250	0.331	2.49	21	3″	21.7	162	1,353	1.8	13	113	3″	53.5	400	3,341	4.5	33	278
4″	4.3	32	268	0.356	2.68	22	4″	22.3	167	1,396	1.9	14	116	4″	54.5	408	3,409	4.5	34	284
5″	4.6	35	287	0.382	2.87	24	5″	23.0	172	1,440	1.9	14	120	5″	55.6	416	3,477	4.6	35	290
6″	4.9	37	306	0.409	3.04	25	6″	23.8	178	1,485	2.0	15	124	6″	56.8	424	3,546	4.7	35	295
7″	5.2	39	325	0.437	3.28	27	7″	24.5	183	1,530	2.0	15	127	7″	57.9	433	3,616	4.8	36	301
8″	5.6	42	349	0.465	3.50	29	8″	25.2	189	1,575	2.1	16	131	8″	59.0	441	3,687	4.9	37	307
9″	5.9	45	371	0.495	3.72	31	9″	26.0	194	1,623	2.2	16	135	9″	60.1	450	3,758	5.0	37	313
10″	6.3	47	394	0.525	3.95	33	10″	26.7	200	1,671	2.2	17	139	10″	61.3	459	3,830	5.1	38	319
11″	6.7	50	418	0.557	4.18	35	11″	27.5	206	1,718	2.3	17	143	11″	62.4	467	3,903	5.2	39	325
3′–0″	7.1	53	442	0.589	4.43	37	6′–0″	28.3	212	1,767	2.4	18	147	9′–0″	63.6	476	3,976	5.3	40	331
1″	7.5	56	467	0.622	4.78	39	1′	29.1	218	1,817	2.4	18	151	1″	64.8	485	4,050	5.4	40	337
2″	7.9	59	494	0.656	4.93	41	2″	29.9	224	1,867	2.5	19	156	2″	66.0	494	4,124	5.5	41	344
3″	8.3	62	519	0.681	5.20	43	3″	30.7	229	1,918	2.6	19	160	3″	67.2	503	4,200	5.6	42	350
4″	8.7	65	546	0.727	5.45	45	4″	31.5	236	1,969	2.6	20	164	4″	68.4	512	4,276	5.7	43	356
5″	9.2	69	573	0.757	5.74	48	5″	32.3	242	2,021	2.7	20	168	5″	69.6	521	4,353	5.8	43	363
6″	9.6	72	601	0.802	6.02	50	6″	33.2	248	2,074	2.8	21	173	6″	70.9	530	4,430	5.9	44	369
7″	10.1	75	630	0.84	6.31	52	7″	34.0	254	2,128	2.8	21	177	7″	72.1	539	4,508	6.0	45	376
8″	10.6	79	660	0.88	6.61	55	8″	34.9	261	2,182	2.9	22	182	8″	73.4	549	4,587	6.1	46	382
9″	11.0	82	690	0.92	6.9	57	9″	35.8	268	2,236	3.0	22	186	9″	74.7	558	4,666	6.2	46	389
10″	11.5	86	721	0.968	7.22	60	10″	36.7	275	2,292	3.1	23	191	10″	75.9	568	4,746	6.3	47	395
11″	12.0	90	753	1.00	7.52	63	11″	37.6	281	2,348	3.1	23	196	11″	77.2	578	4,827	6.4	48	402
4′–0″	12.6	94	786	1.0	8	65	7′–0″	38.5	288	2,405	3.2	24	200	10′–0″	79	590	4,910	7	49	410
1″	13.1	98	819	1.1	8	68	1″	39.4	295	2,463	3.3	25	205	3″	82	620	5,160	7	50	430
2″	13.6	102	853	1.1	8	71	2″	40.3	302	2,521	3.4	25	210	6″	87	650	5,410	7	50	450
3″	14.2	106	887	1.2	9	74	3″	41.3	309	2,580	3.4	26	215	9″	91	680	5,670	8	60	470
4″	14.7	110	922	1.2	9	77	4″	42.2	316	2,640	3.5	26	220	11′–0″	95	710	5,940	8	60	490
5″	15.3	115	958	1.3	10	80	5″	43.2	323	2,700	3.6	27	225	3″	99	740	6,210	8	60	520
6″	15.9	119	994	1.3	10	83	6″	44.2	330	2,761	3.7	27	230	6″	104	780	6,490	9	70	540
7″	16.5	123	1,031	1.4	10	86	7″	45.2	338	2,823	3.8	28	235	9″	108	810	6,770	9	70	560
8″	17.1	128	1,069	1.4	11	89	8″	46.2	346	2,885	3.8	29	240	12′–0″	113	850	7,070	9	70	590
9″	17.7	133	1,108	1.5	11	92	9″	47.2	353	2,948	3.9	29	246	3″	118	880	7,370	10	70	610
10″	18.3	137	1,147	1.5	11	96	10″	48.2	360	3,012	4.0	30	251	6″	123	920	7,670	10	80	640
11″	19.0	142	1,187	1.6	12	99	11″	49.2	368	3,076	4.1	31	256	9″	128	950	7,980	11	80	670

* Applies to pipes also

† Numbers in these columns are also areas in sq ft. corresponding to diam.

Table 213. Cylindrical Tanks—Capacity for Depth or Length of 1 ft. and 1 in. Cu. ft., Gals., and Lbs.—(*Continued*)

Depth Diam.	1 Ft. Cu. ft *	1 Ft. Gals.	1 Ft. Lbs.	1 In. Cu. ft	1 In. Gals	1 In. Lbs	Depth Diam.	1 Ft Cu. ft. *	1 Ft Gals.	1 Ft Lbs.	1 In. Cu. ft	1 In. Gals.	1 In. Lbs.	Depth Diam.	1 Ft Cu. ft *	1 Ft Gals.	1 Ft Lbs.	1 In. Cu. ft.	1 In. Gals.	1 In. Lbs.
13'-0"	133	990	8 290	11	80	690	21'-6"	365	2,700	22,700	30	220	1,900	37'-0"	1075	8,050	67,200	90	670	5600
3"	138	1,030	8,620	11	90	720	22'-0"	380	2,850	23,800	30	240	2,000	6"	1105	8,250	69,000	90	690	5800
6"	143	1,070	8,940	12	90	750	6"	400	3,000	24,900	35	250	2,100	38'-0"	1135	8,500	70,900	95	710	5900
9"	148	1,110	9,280	12	90	770	23'-0"	415	3,100	25,900	35	260	2,200	6"	1165	8,700	72,800	95	730	6100
14'-0"	154	1,150	9,620	13	100	800	6"	435	3,250	27,100	35	270	2,300	39'-0"	1195	8,950	74,700	100	750	6200
3"	159	1,190	9,970	13	100	830	24'-0"	450	3,400	28,200	35	280	2,400	6"	1225	9,150	76,600	100	760	6400
6"	165	1,240	10,320	14	100	860	6"	470	3,500	29,400	40	290	2,500	40'-0"	1255	9,400	78,600	105	780	6500
9"	171	1,280	10,680	14	110	890	25'-0"	490	3,650	30,700	40	300	2,600	6"	1290	9,650	80,500	105	800	6700
15'-0"	177	1,320	11,040	15	110	920	6"	510	3,800	31,900	40	320	2,700	41'-0"	1320	9,850	82,500	110	820	6900
3"	183	1,370	11,420	15	110	950	26'-0"	530	3,950	33,200	45	330	2,800	6"	1355	10,150	84,600	115	850	7100
6"	189	1,410	11,790	16	120	980	6"	550	4,150	34,500	45	350	2,900	42'-0"	1385	10,350	86,600	115	860	7200
9"	195	1,460	12,180	16	120	1,010	27'-0"	575	4,300	35,700	50	360	3,000	6'	1420	10,600	88,700	120	880	7400
16'-0"	201	1,500	12,570	17	120	1,050	6"	595	4,450	37,100	50	370	3,100	43'-0"	1450	10,850	90,800	120	900	7600
3"	207	1,550	12,960	17	130	1,080	28'-0"	615	4,600	38,500	50	380	3,200	6"	1485	11,100	92,900	125	930	7700
6"	214	1,600	13,360	18	130	1,110	6"	640	4,750	39,900	55	390	3,300	44'-0"	1520	11,400	95,100	125	950	7900
9"	220	1,650	13,770	18	140	1,150	29'-0"	660	4,950	41,300	53	410	3,400	6"	1555	11,650	97,200	130	970	8,100
17'-0"	227	1,700	14,190	19	140	1,180	6"	685	5,100	42,700	55	430	3,600	45'-0"	1590	11,900	99,400	135	990	8,300
3"	234	1,750	14,610	20	150	1,220	30'-0"	705	5,300	44,200	60	440	3,700	6"	1625	12,150	101,600	135	1010	8,500
6"	240	1,800	15,030	20	150	1,250	6"	730	5,450	45,700	60	450	3,800	46'-0"	1660	12,450	103,900	140	1040	8,700
9"	247	1,850	15,470	21	150	1,290	31'-0"	755	5,650	47,200	65	470	3.900	6"	1700	12,700	106,100	140	1060	8,800
18'-0"	254	1,900	15,910	21	160	1,330	6"	780	5,850	48,700	65	490	4,100	47'-0"	1735	13,000	108,400	145	1080	9,000
3"	262	1,960	16,350	22	160	1,360	32'-0"	805	6,000	50,300	65	500	4,200	6"	1770	13,250	110,800	150	1100	9,200
6"	269	2,010	16,800	22	170	1,400	6"	830	6,200	51,900	70	520	4,300	48'-0"	1810	13,550	113,100	150	1130	9,400
9"	276	2,060	17,260	23	170	1,440	33'-0"	855	6,400	53,400	70	530	4,400	6"	1845	13,800	115,400	155	1150	9,600
19'-0"	284	2,120	17,720	24	180	1,480	6"	880	6,600	55,100	75	550	4,600	49'-0"	1885	14,100	117,900	155	1170	9,800
3"	291	2,180	18,190	24	180	1,520	34'-0"	910	6,800	56,800	75	570	4,700	6"	1925	14,400	120,200	160	1200	10,000
6"	299	2,230	18,670	25	190	1,560	6"	935	7,000	58,400	80	580	4900	50'-0"	1965	14,700	122,700	165	1230	10,200
9"	306	2,290	19,150	25	190	1,600	35'-0"	960	7,200	60,100	80	600	5000							
20'-0"	315	2,350	19,700	25	200	1,600	6"	990	7,400	61,900	85	620	5200							
6"	330	2,450	20,600	30	200	1,700	36'-0"	1020	7,650	63,600	85	640	5300							
21'-0"	345	2,600	21,600	30	220	1,800	6"	1045	7,800	65,400	85	650	5500							

1 cu. ft. = 7.48 gals. and 62.5 lbs. in above tables. Errors of about 2 per cent, due to rounding number.

* Numbers in these columns are also areas in sq ft corresponding to diam.

Table 214. Tanks—Capacities in Cubic Feet, Gallons and Pounds, for Depths of 1 ft. and 1 in., for Horizontal Areas in Square Feet*

Maximum error due to round numbers = 0.26 per cent.

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft.	Gals.	Weight, lbs	Cu. ft.	Gals.	Weight, lbs.
0 1	0.1	0.748	6.25	0 0083	0.06	0.52
0 2	0 2	1.5	12 5	0 017	0.12	1.04
0 3	0.3	2 2	18 7	0 025	0 19	1.56
0.4	0.4	3 0	25 0	0 033	0.25	2.08
0 5	0.5	3 7	31.2	0 042	0.31	2.60
0 6	0.6	4 5	37.5	0 050	0.37	3.12
0.7	0.7	5.2	43.7	0 058	0.44	3.65
0.8	0.8	6.0	50 0	0 067	0.50	4.17
0 9	0.9	6 7	56.2	0 075	0.56	4.69
1.0	1 0	7 5	62.5	0.083	0.62	5.21
1.1	1 1	8 2	69	0.092	0.68	5.7
1.2	1 2	9 0	75	0.100	0.75	6.2
1.3	1.3	9.7	81	0 108	0.81	6.8
1.4	1.4	10.5	87	0 117	0.87	7.3
1.5	1 5	11 2	94	0 125	0.93	7.8
1.6	1 6	12 0	100	0.133	1.00	8.3
1.7	1 7	12.7	106	0 142	1.06	8 9
1.8	1.8	13.5	112	0 150	1.12	9.4
1.9	1 9	14 2	119	0 158	1.18	9.9
2.0	2 0	15 0	125	0.167	1.25	10.4
2.1	2 1	15 7	131	0 175	1.31	10 9
2.2	2 2	16 4	137	0 183	1.37	11 5
2 3	2 3	17 2	144	0 192	1.44	12 0
2 4	2.4	17 9	150	0 200	1.50	12 5
2.5	2 5	18.7	156	0 208	1.56	13.0
2.6	2 6	19.4	162	0 217	1.62	13 5
2 7	2 7	20 2	169	0 225	1 69	14 0
2 8	2 8	20 9	175	0 233	1 75	14 6
2 9	2 9	21 7	181	0 242	1 81	15 1
3 0	3.0	22.4	187	0 250	1.87	15 6
3.1	3.1	23 2	194	0 258	1.93	16 1
3.2	3.2	23 9	200	0 267	1.99	16 6
3.3	3 3	24 6	206	0.275	2.06	17 2
3.4	3.4	25 4	212	0.283	2.12	17.7
3.5	3 5	26 2	219	0.292	2.18	18.2
3.6	3 6	26 9	225	0.300	2.24	18.7
3.7	3.7	27.6	231	0.308	2 31	19.3
3.8	3.8	28 4	237	0 317	2 37	19 8
3 9	3 9	29 2	244	0 325	2 43	20 3
4 0	4 0	29 9	250	0 333	2 49	20.8
4.1	4.1	30.7	256	0.342	2.55	21.3
4.2	4.2	31 4	262	0 350	2 61	21.9
4 3	4.3	32 1	269	0 358	2 68	22.4
4.4	4 4	32 9	275	0 367	2 74	22 9
4 5	4 5	33 6	281	0 375	2 80	23 4
4 6	4 6	34 4	287	0 383	2.86	23 9
4 7	4 7	35 1	294	0 392	2 93	24 5
4.8	4 8	35 9	300	0 400	2 99	25.0
4 9	4 9	36.6	306	0.408	3.05	25.5
5 0	5.0	37 4	312	0 417	3.12	26 0
5 1	5 1	38 1	319	0.425	3 18	26 5
5 2	5.2	38 9	325	0 433	3 24	27 0
5 3	5 3	39 6	331	0.442	3.31	27 6
5 4	5.4	40 4	337	0.450	3 37	28.1
5.5	5.5	41.1	344	0.458	3.43	28.6
5.6	5 6	41 9	350	0.467	3 49	29.1
5 7	5 7	42 6	356	0 475	3.56	29.7
5 8	5 8	43.4	362	0.483	3 62	30 2
5.9	5 9	44.1	369	0.492	3.68	30.7
6.0	6.0	44.9	375	0 500	3.74	31 2
6.1	6.1	45.6	381	0.508	3.80	31.7
6.2	6 2	46.4	387	0.517	3 86	32.3
6 3	6 3	47.1	394	0.525	3 93	32.8
6.4	6.4	47.9	400	0.533	3.99	33.3

* Useful also for pipes per ft. or in. of length.

Table 214. Tanks—Capacity for Depths of 1 ft. and 1 in.—(*Continued*)

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft	Gals.	Weight, lbs	Cu ft	Gals.	Weight, lbs
6 5	6.5	48 6	406	0.542	4 05	33 8
6 6	6.6	49 4	412	0 550	4 11	34 3
6 7	6.7	50 1	419	0 558	4 18	34 9
6 8	6.8	50 9	425	0 567	4 24	35 4
6 9	6 9	51 6	431	0 575	4 30	35 9
7 0	7 0	52 4	437	0 583	4 36	36 5
7 1	7 1	53 1	444	0 592	4 42	37 0
7.2	7.2	53 8	450	0.600	4 48	37 5
7 3	7 3	54 6	456	0 608	4 55	38 0
7 4	7 4	55 3	462	0 617	4 61	38.5
7.5	7.5	56.1	469	0.625	4 67	39 1
7 6	7 6	56 8	475	0 633	4 73	39 6
7 7	7 7	57 6	481	0 642	4 80	40 1
7 8	7 8	58 3	487	0 650	4 86	40 6
7 9	7 9	59 1	494	0 658	4 92	41 1
8.0	8.0	59 8	500	0.667	4 99	41 7
8 1	8 1	60 6	506	0 675	5 05	42 2
8 2	8.2	61 3	512	0 683	5 11	42 7
8 3	8 3	62 1	519	0 692	5 18	43 2
8 4	8.4	62 8	525	0 700	5 24	43 7
8 5	8 5	63 6	531	0 708	5 30	44 3
8 6	8 6	64 3	537	0 717	5 36	44.8
8 7	8.7	65.1	544	0 725	5.43	45.3
8.8	8 8	65 8	550	0.733	5 49	45 8
8 9	8 9	66 6	556	0 742	5 55	46 3
9 0	9 0	67 3	562	0 750	5 61	46 9
9 1	9 1	68 1	569	0 758	5 67	47 4
9 2	9 2	68 8	575	0 767	5 73	47 9
9 3	9 3	69 6	581	0 775	5 80	48 4
9 4	9 4	70 3	587	0 782	5 86	49 0
9 5	9 5	71 1	594	0 792	5 92	49 5
9 6	9 6	71 8	600	0 800	5.98	50 0
9 7	9 7	72 6	606	0 808	6 05	50 5
9 8	9 8	73 3	612	0 817	6 11	51 0
9 9	9 9	74 1	619	0 825	6 17	51 6
10 0	10 0	74 8	625	0 833	6 23	52 0
20 0	20 0	150	1,250	1 67	12 5	100
30 0	30 0	224	1,870	2 50	18 7	160
40 0	40 0	299	2,500	3 33	24 7	210
50.0	50 0	374	3,120	4 17	31 2	260
60 0	60 0	449	3,750	5 00	37 4	310
70 0	70 0	524	4,370	5 83	43 6	360
80 0	80 0	598	5,000	6 67	49 8	420
90 0	90 0	673	5,620	7 50	56 1	470
100 0	100 0	748	6,250	8 33	62 3	520
110 0	110 0	823	6,870	9 17	68 5	570
120 0	120 0	898	7,500	10 00	74 8	620
130 0	130.0	972	8,120	10 83	81 0	680
140 0	140.0	1,047	8,750	11 67	87 0	730
150 0	150 0	1,122	9,380	12 50	93 4	780
160.0	160.0	1,197	10,000	13 33	99.7	830
170 0	170.0	1,272	10,600	14 17	105.9	890
180 0	180.0	1,346	11,250	15 00	112.1	940
190 0	190.0	1,421	11,850	15 83	118 4	990
200.0	200.0	1,496	12,500	16 67	124.6	1,040
210.0	210.0	1,570	13,100	17 5	131	1,090
220.0	220 0	1,650	13,750	18 3	137	1,150
230 0	230.0	1,720	14,350	19 2	144	1,200
240.0	240.0	1,800	15,000	20 0	150	1,250
250 0	250.0	1,870	15,600	20.8	156	1,300
260 0	260.0	1,950	16,250	21.7	162	1,350
270 0	270.0	2,020	16,850	22.5	169	1,410
280 0	280.0	2,090	17,500	23.3	175	1,460
290 0	290.0	2,170	18,100	24 2	181	1,510
300.0	300.0	2,240	18,750	25.0	187	1,560
310 0	310.0	2,350	19,400	25 8	193	1,610
320 0	320.0	2,390	20,000	26 7	199	1,660
330 0	330.0	2,470	20,600	27 5	206	1,720
340 0	340.0	2,540	21,250	28 3	212	1,770

Continued on next page.

Table 214. Tanks—Capacity for Depths of 1 ft. and 1 in.—(*Concluded*)

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft.	Gals.	Weight, lbs	Cu. ft.	Gals.	Weight, lbs
350.0	350.0	2,620	21,850	29.2	218	1,820
360.0	360.0	2,690	22,500	30.0	224	1,870
370.0	370.0	2,770	23,100	30.8	231	1,930
380.0	380.0	2,840	23,750	31.7	237	1,980
390.0	390.0	2,920	24,350	32.5	243	2,030
400.0	400.0	2,990	25,000	33.3	249	2,080
410.0	410.0	3,070	25,600	34.2	255	2,130
420.0	420.0	3,140	26,250	35.0	261	2,190
430.0	430.0	3,220	26,850	35.8	268	2,240
440.0	440.0	3,290	27,500	36.7	274	2,290
450.0	450.0	3,360	28,100	37.5	280	2,340
460.0	460.0	3,440	28,750	38.3	286	2,390
470.0	470.0	3,510	29,400	39.2	293	2,450
480.0	480.0	3,590	30,000	40.0	299	2,500
490.0	490.0	3,660	30,600	40.8	305	2,550
500.0	500.0	3,750	31,250	41.7	310	2,600
510.0	510.0	3,800	31,850	42.5	320	2,650
520.0	520.0	3,900	32,500	43.3	325	2,700
530.0	530.0	3,950	31,100	44.2	330	2,760
540.0	540.0	4,050	33,750	45.0	335	2,810
550.0	550.0	4,100	34,350	45.8	345	2,860
560.0	560.0	4,200	35,000	46.7	350	2,910
570.0	570.0	4,250	35,600	47.5	355	2,960
580.0	580.0	4,350	36,250	48.3	360	3,020
590.0	590.0	4,400	36,850	49.2	370	3,070
600.0	600.0	4,500	37,500	50.0	375	3,120
610.0	610.0	4,550	38,100	50.8	380	3,170
620.0	620.0	4,650	38,750	51.7	385	3,230
630.0	630.0	4,700	39,400	52.5	395	3,280
640.0	640.0	4,800	40,000	53.3	400	3,330
650.0	650.0	4,900	40,600	54.2	405	3,380
660.0	660.0	4,950	41,250	55.0	410	3,430
670.0	670.0	5,000	41,850	55.8	420	3,490
680.0	680.0	5,100	42,500	56.7	425	3,540
690.0	690.0	5,150	43,100	57.5	430	3,590
700.0	700.0	5,250	43,750	58.3	435	3,650
710.0	710.0	5'300	44,350	59.2	445	3,700
720.0	720.0	5,400	45,000	60.0	450	3,750
730.0	730.0	5,450	45,600	60.8	455	3,800
740.0	740.0	5,550	46,250	61.7	460	3,850
750.0	750.0	5,600	46,850	62.5	470	3,910
760.0	760.0	5,700	47,500	63.3	475	3,960
770.0	770.0	5,750	48,100	64.2	480	4,010
780.0	780.0	5,850	48,750	65.0	485	4,060
790.0	790.0	5,900	49,400	65.8	490	4,110
800.0	800.0	6,000	50,000	66.7	500	4,170
810.0	810.0	6,050	50,600	67.5	505	4,220
820.0	820.0	6,150	51,250	68.3	510	4,270
830.0	830.0	6,200	51,850	69.2	515	4,320
840.0	840.0	6,300	52,500	70.0	525	4,370
850.0	850.0	6,350	53,100	70.8	530	4,430
860.0	860.0	6,450	53,750	71.7	535	4,480
870.0	870.0	6,500	54,350	72.5	540	4,530
880.0	880.0	6,600	55,000	73.3	550	4,580
890.0	890.0	6,650	55,650	74.2	555	4,630
900.0	900.0	6,750	56,250	75.0	560	4,690
910.0	910.0	6,800	56,850	75.8	565	4,740
920.0	920.0	6,900	57,500	76.7	575	4,790
930.0	930.0	6,950	58,100	77.5	580	4,840
940.0	940.0	7,000	58,750	78.3	585	4,900
950.0	950.0	7,100	59,400	79.2	590	4,950
960.0	960.0	7,200	60,000	80.0	600	5,000
970.0	970.0	7,250	60,650	80.8	605	5,050
980.0	980.0	7,350	61,250	81.7	610	5,100
990.0	990.0	7,400	61,850	82.5	615	5,160
1,000.0	1,000.0	7,500	62,500	83.3	625	5,200

Above table is based on 1 cu. ft. of water weighing 62.5 lbs and containing 7.48 gals

Example of use: Cistern 18′ × 50′ × 10′1″ deep contains, how many gals.? 18 × 50 = 900 Entering table under "Area" at 900, find 6750 gals. per ft. depth; 560 per in. Capacity = 67,500 + 560 = 68,060 gals The above weights are exclusive of weight of tank. For circular tanks obtain areas from tables of circles or compute.

CHAPTER XXXIX

MISCELLANY

Cold Periods in U. S.

(Bull. P., 1906, U. S. Weather Bureau)

1717. "Great Snow," Feb. 19–24; 6 ft. deep, Boston.

1741. Intense cold. Long Island Sound frozen. Connecticut river ice would carry horses, April 1.

1765–6. Hudson frozen over at Paulus Hook. 26° F., St. Johns river, Fla.

1780 Coldest, except 1856. Chesapeake bay frozen over from head to Potomac; −6° at Williamsburg, Va. Ice solid enough to support pedestrians. Ice entirely between New York and Staten Island, and on Long Island Sound. Troops crossed from New Jersey to Staten Island on ice. Bayou St. John, New Orleans, was frozen. Delaware river frozen over from Dec. 1 to Mar. 14, ice 2 or 3 ft. thick.

1783. −12° at Philadelphia; Hartford, Conn., −20°.

1787–8. Ground frozen at Savannah, 20° F.

1792. Ohio river at Wheeling frozen over for 40 days.

1796–7. Ohio and Mississippi rivers and confluents frozen to their junction; 17° Charleston; −19° Cincinnati.

1800. Snow 18 in. deep in Georgia. Snowfall 5 in. deep at St. Mary's river, Fla.. Natchez, Miss., 6°. Sleet and heavy frosts in Louisiana.

1816. At Williamstown, Mass., frost in each summer month. Snow, June 8 in Vermont. Frost in Philadelphia, July.

1820–1. Hudson at Paulus Hook covered with ice (fourth time in 100 yrs).

1825–6. −27°, Portland, Me.; −24°, Amherst; −18°, Springfield; 0°, Washington; 14° in Louisiana; Montreal, −38°.

1831. Mississippi river frozen for 130 mi. below mouth of Ohio.

1835. Long Island Sound closed; Boston harbor, nearly so.

1835. Jan. 4, Bangor, Me., −40°; Bath, Me., −40°; Portland, Me., −21°; Montpelier, −40°; Concord, N. H., −35°; Boston, −15°; Pittsfield, Mass., −32°; Albany, −32°; Poughkeepsie, −35°; New York, −5°; Lancaster, Pa., −22°; Washington, −16°; Baltimore, −10°. Jan. 5, New Haven, Conn., −27°; Providence, −16°; Philadelphia, −6°. Feb. 8, Chicago, −22°; St. Louis, −25°; Cincinnati, −18°; Nashville, −10°; Huntsville, Ala., −9°; Natchez, Miss, 0°; Baton Rouge, 10°; Jacksonville, 8°; Richmond, −6°; Savannah, 3°; Augusta, −2°.

1851–2. St. Louis, −14°; New Orleans, 17°; Pensacola, 10°; Washington, −7°; New York, −8°. Potomac at Washington frozen over for 3 weeks. East river frozen; crossed from Jan. 20–24.

1854. Salt Lake City, −14°; Fort Dallas, Ore., −15°; Fort Defiance, N. Mex., −20°; San Francisco, 27°.

1856. Pittsburgh, −18°. Ice formed in Louisiana Severest winter of 50 yrs. in Mississippi valley.

1864. Cincinnati, −5°; Fort Laramie, Wy., −40°.

Cold Periods in U. S.—(*Continued*).

1867. December. −10°, Wytheville, Va.; St. Paul, −39°.
1872. December. Chicago, −23°; Lunenburg, Vt., −45°;
1875. January. Fort Garland, Col., −40°; Logansport, Ind., −30°; New Haven, −14°.
1875. January 9. −6°, New York; −12°, Pittsburgh; −3°, Washington.
1904–5. 6° to 8° below normal for Eastern States.

Breaking up of Ice. To rupture ice, Asst. Eng. C. R. Adams, U. S. G. S., from experimental data, computes following stream gage hights required.

River width, ft.	Ice thickness, in.	Change in gage hight, ft
500	6	0.0005
500	12	0 0015
500	24	0.0077
500	36	0 0172
200	6	0 003
200	12	0 012
200	24	0 048
100	6	0.012
100	12	0 048
100	24	0 191
50	6	0 048
50	12	0 191
50	24	0 766

Tensile strength taken as 160 lbs. per sq in., shear, 86 lbs. per sq. in. E. N., Feb. 2, 1911.

Non-freezing Mixtures. A glycerin solution of 3.5 lbs. per gallon of water will not freeze at + 10° F.; 5.25 lbs. at − 10° F. Glycerin has no effect upon metals, but disintegrates rubber; the continued use of glycerin in water renders it liable to decomposition, developing compounds having corrosive action on metals. Denatured alcohol does not injure metal or rubber; a 50 per cent. solution is inflammable; 20 per cent. solution will withstand − 10°; 30 per cent., − 15°; 40 per cent., − 20°; 50 per cent., − 35°.

Table 215. Freezing Points, Degrees F., of Solutions of Salt (Sodium Chloride) and Calcium Chloride

Pounds per gal	Sodium chloride	Calcium chloride
0 5	+24	+29
1.0	+18	+27
1 25	+15	
1 5	+12	+23
1.75	+ 9	
2.0	+ 6	+18
2 25	+ 3	
2 5	+ 1	+ 4
3.0	− 3	
3 5	− 8	+ 6
4.0		+17
4 5		+27
5 0		+39
5.5		+54

Sodium chloride corrodes steel tanks and barrel hoops. Wooden barrels for calcium chloride should be coated with asphalt to prevent leakage when the staves shrink.—("Quarterly" of National Fire Protection Assn., 1911.)

Lamé's general formula for thick hollow cylinder:

$$s = \left[r_1^2 P_1 - r_2^2 P_2 + \frac{r_1^2 r_2^2}{x^2}(P_1 - P_2)\right] \div (r_2^2 - r_1^2)$$

s = tangential unit stress at distance x from axis of cylinder
r_1 = internal radius of cylinder
r_2 = external radius of cylinder
P_1 = internal unit stress
P_2 = external unit stress.

Earthquakes—Effects on W. W. Structures. Committee of engineers, San Francisco, Apr., 1906. Conclusions from extended observations of damage done: 1. Greater attention should be given to avoidance of threatening geologic faults, in locating important waterworks structures. 2. Skillfully designed and well built earth dams were proven of great stability, and deserving increased confidence. 3. Concrete dams of gravity section are able to withstand shocks of great severity without damage. 4. Distributing reservoirs, pumping machinery, elevated tanks and standpipes, if secure as to foundations and intelligently designed according to best practice, will withstand shocks sufficient to destroy most buildings. 5. Pipes or conduits of any character are almost certain to fail when intersected by a plane of large movement, whether faulting, sliding or settling. In planning distributing systems for cities, important supply pipes should be located along routes carefully selected for stability; areas liable to serious disturbances should be so segregated as to be quickly separable from remainder of system. 6. Distributing pipe systems should have abundance of suitably placed, well-maintained valves, conspicuously marked in field and clearly recorded, for isolating pipes and sections; critically important supply mains should be duplicated along widely separated routes; ample supplies of water should be maintained close to centers of use; structures liable to shock should be of substantial types.

Near San Francisco, are several of greatest *earth dams* in the world. As permanent longitudinal displacement along a fault in this region was 6 to 7 ft., all these dams must have been shaken terrifically; 2 were directly on fault line. Most were wholly uninjured or but slightly injured. At San Andreas dam, the fault passed through the east end, where a natural ridge was made part of the dam; one crack, without injury, intersected the concrete inlet culvert with embedded railroad iron. Brick-lined tunnel from waste overflow, intersected by the fault, was fractured and offset. Main body of dam showed a crack 2 to 3 in. wide, longitudinally, its entire length, along the center line, also some slight transverse cracks. No leakage resulted; the earth structure may be considered wholly uninjured.

In North dam, Saratoga reservoir, San Jose Water Co., traversed by a fault transversely, the 10-in. cast-iron outlet pipe was shattered and a joint near toe of dam pulled apart, thus emptying the reservoir. At San Leandro

dam, the reservoir was full to overflowing; the shock caused waves 3.5 ft. high; no injury whatever was done. Piedmont dam, only a few months old, 260 ft. long on top, maximum hight 65 ft., both slopes 1 on 2, water slope paved with 6-in. concrete in squares, with asphaltum joints; this reservoir, just filled first time, had a rough shaking which caused 6-in. settlement at center and a few small transverse and longitudinal cracks near one end; no cracking of the concrete occurred nor material injury of any character. All dams mentioned were built by spreading in thin layers, wetting and rolling.

Only 2 *masonry dams* (concrete) are in this vicinity, San Mateo and Partola, each a few hundred feet from the fault line and parallel to it. Neither showed evidence of injury in any particular.

Of numerous *distributing reservoirs*, mostly lined with brick, concrete, concrete and asphalt, or clay puddle protected with paving stones, only Lake Honda, 33 mg., in San Francisco, was injured; practically wholly in excavation, with a lining of various kinds of masonry, it was broken so as to need repairs. Slope on west side is of very sandy soil supported by a retaining wall, consisting of brick buttresses with horizontal brick arches between, and constituting the side of basin; this wall ruptured and failed by shearing near the bottom of the reservoir, although irregular cracks in one place extended to top.

Many *pumping stations* of corrugated steel on wood frames and of brick; machinery and boilers on good masonry foundations, in some cases on piles; individual guyed, steel stacks, with 2 or 3 exceptions of brick, did not suffer any material damage affecting serviceability, although some brick walls were damaged. Brick stacks were in most cases broken off.

Many *elevated water tanks and standpipes* connected with public works escaped with little or no injury, but numerous low tanks for road sprinkling and locomotive supply were thrown down. Important elevated tanks at Santa Clara were wrecked; the shake was especially severe here. Tower 80 ft. high, divided into 3 stories by horizontal channel or I-beam struts, made up of 4 rows of 4 Z-bar columns, carrying 4 rows of plate girders, on which were 15-in. I-beams forming floor system; plank floor; 4 cylindrical wood tanks, aggregate capacity 180,000 gals., contained during the quake 125,000 gals. Structure swayed violently; tanks lifted from setting and moved south, 2 finally being thrown to ground 75 ft. from tower; 2 north tanks were on the extreme south portion of floor at end of quakes; structure ceased to sway and stood upright. Movement of tanks had overturned some floor beams; they bent downward, ends slipped from supports and let tanks drop, carrying lateral bracing and pulling down most columns. On north side many columns stood intact with lateral and sway bracing uninjured, except top story. Structure was rebuilt along original lines with additional bracing between girders to resist overturning, and one large steel tank anchored to I-beams.

Pipe lines in firm ground sustained no considerable damage; the injuries were done in marshy, filled, and soft alluvial soils. If such danger spots cannot be wholly avoided for important mains, partial or entire immunity may be found in pile foundations extending well below compressible soil, and

heavy flexible joints across lines of probable separation.—(T. A. S. C. E., Vol. 59, 1907.)

Table 216. Life of Waterworks Structures

Compiled from citations by experts in evaluation suits.—(Leonard Metcalf, J. N. E. W. W. Assn., 1910)

Structure	Life, yrs.	Structure	Life, yrs.
Reservoirs..........	50 to 100	Masonry conduits...	50 to 100
Steel standpipes......	25 to 40	Cast-iron pipe, large, non-tuberculating waters	60 to 100
Masonry buildings..	30 to 50	Cast-iron pipe, tuberculating waters	50 to 75
Wooden buildings ..	20 to 40	Cast-iron pipe, small	30 to 50
Filter plants, permanent construction	25 to 50	Steel pipe,* no concrete jacket	25 to 40
Pumping machinery, high duty, large units	30 to 50	Wood-stave pipe.	20 to 30
Ordinary pumping machinery	20 to 30	Wrought-iron service pipe	15 to 30
Steam engines..	15 to 25	Hydrants..........	35 to 50
Boilers	12 to 20	Valves.............	40 to 50
Electric generators and motors	20 to 30	Meters............	15 to 25

* 30 years assumed in Catskill aqueduct studies.

Appraisal of Plants. In appraising water companies, the going-concern value should be considered, based on the fact that the plants are in actual connection with paying customers, and that if purchased, full income would start from day of purchase. Whereas a reproduction of the existing plant, while costing the same, would have to wait one or more years for a normal demand for water.*

Deductions from Cost of Water because of Quality. Hardness, normal, 50 parts per million; deduct 10 cts. per million gallons for each part per million of hardness in excess of 50. Traces of iron and salt, due to overdraft, reduce the commercial value of water; assume $10 per million gallons. For heavy population on the watershed, deduct $20 per million gallons.—(Report of Commission on Staten Island Water, New York, Feb. 8, 1906.)

Water-level recording apparatus for indicating and recording changes of hight at a distant point has been made for many years by Geo. E. Winslow, Waltham, Mass., to show hight of gas-holders, etc., or depth of water in reservoirs, tanks, standpipes, ponds, rivers, canals, etc. Hight is electrically transmitted by primary instrument, or transmitter, to indicating and recording instrument, or indicator, which may be placed in a superintendent's office or elsewhere, thus giving the person in charge a correct idea at any and all times, and recording on a paper dial making one revolution in 7 days. The dials, kept on file, will settle questions of past readings. Consumption by day or night can also be readily computed. This apparatus acts as a watchman and as a check to reports of a watchman, and notifies the person in charge when any unusual consumption has taken place. Contacts to ring a

* The reader is referred to following in Trans. Am Soc C E. "The Going Value of Waterworks," Vol. 73, 1911. "The Valuation of Public Service Corporation Property," Vol 72, 1911 "Valuation of Waterworks Property," Vol. 38, 1897. "Valuation and Fair Rates in the Light of the Maine Supreme Court decisions in the Waterville and Brunswick Cases," Vol 64, 1909.

Table 217. Cost of Cubic Feet of Water at Stated Rates per 1000 Gals.

No of cu. ft.	Cost per 1,000 Gals. (For cost per mg. move decimal point 3 places to right)							
	5 Cts.	6 Cts.	8 Cts.	10 Cts.	15 Cts	20 Cts.	25 Cts.	30 Cts
20	$0 007	$0 009	$0 012	$0 015	$0.021	$0 030	$0 037	$0.045
40	0 015	0 018	0.024	0.030	0.045	0.060	0.075	0.090
60	0.022	0.027	0.036	0.045	0.066	0.090	0.112	0.135
80	0 030	0.036	0.048	0.060	0.090	0.120	0.150	0.180
100	0.037	0.049	0.060	0.075	0.111	0.150	0.187	0 224
200	0.075	0.090	0.120	0.150	0.225	0.299	0.374	0 449
300	0 112	0.135	0.180	0.224	0.336	0.449	0.561	0.673
400	0.150	0.180	0.239	0.299	0.450	0.598	0.748	0 898
500	0.188	0.224	0.299	0.374	0.564	0.748	0.935	1.122
600	0.224	0.269	0.359	0.449	0.674	0.898	1.122	1.346
700	0 262	0.314	0.419	0.524	0.786	1.047	1.309	1.571
800	0.299	0.350	0.479	0.598	0.897	1.197	1.496	1.795
900	0.337	0.404	0.539	0.673	1.011	1.346	1.683	2 020
1,000	0.374	0.449	0.598	0.748	1.122	1.496	1.870	2.244
2,000	0.748	0.898	1.197	1.496	2.244	2.992	3.740	4 488
3,000	1.122	1.346	1.795	2.244	3.366	4.488	5.610	6 732
4,000	1.496	1.795	2.393	2 992	4.488	5.984	7.480	8 976
5,000	1 870	2.244	2.992	3 740	5.610	7 480	9.350	11 220
6,000	2.244	2.692	3.590	4.488	6.732	8 976	11.220	13 464
7,000	2.618	3.141	4.189	5.236	7.854	10.472	13.090	15 708
8,000	2 992	3.590	4.787	5.984	8.976	11 968	14.961	17 953
9,000	3.366	4.039	5.385	6.732	10.098	13 464	16.831	20 197
10,000	3.74	4.488	5.984	7.480	11.122	14 961	18.701	22 441
20,000	7.48	8.976	11.968	14 961	22.443	29.992	37.402	44 882
30,000	11.22	13.46	17.95	22.44	33.664	44.88	56.10	67 32
40,000	14 96	17.95	23.94	29 92	44.885	59.84	74.80	89 77
50,000	18.70	22.44	29 92	37 40	56.103	74 80	93.50	112.20
60,000	22.44	26 92	35 90	44 88	67.323	89 76	112 20	134 64
70,000	26.18	31 41	41 89	52 36	78.543	104 72	130 90	157 08
80,000	29.92	35 90	47.87	59.84	89.766	119.68	149.61	179.53
90,000	33.66	40 39	53 85	67 32	100.986	134 64	168.31	201 97
100,000	37.40	44 88	59.84	74 80	111.22	149 61	187 01	224 41
200,000	74 8L	89 76	119 68	149 61	224 43	299.22	374 02	448 82
300,000	112.20	134 64	179 53	224 41	336.63	448 83	561.03	673 24
400,000	149.61	179.53	239.37	299.22	448.85	598 44	748.05	897 66
500,000	187.01	224.41	299.22	374 02	561 03	748 05	935.06	1,122.07
600,000	224.41	269 29	359 06	448.82	673 23	897.66	1,122 07	1,346 49
700,000	261.81	314 18	418 90	523.63	785 43	1,047.27	1,309.08	1,570 88
800,000	299.22	359 06	478 75	598 44	897 66	1,196.88	1,496 10	1,795 32
900,000	336 62	403 94	538 59	673 24	1,009 86	1,346.49	1,683 11	2,019 73
1,000,000	374 02	448 83	598 44	748.05	1,122.06	1,498 10	1,870 12	2,244 15

bell when a desired point has been reached can be put in and operated independently of the indicator at small additional expense. Indicator is made to indicate and record graduations as ordered and will indicate within two-thirds of one graduation. Care should be taken that graduations be large enough to cover surge of the water where the transmitter is located. Standard found most practical for reservoirs and standpipes is as follows:

Graduations	Maximum range	Graduations	Maximum range
$\frac{1}{10}$ ft.	12 ft.	4 in.	40 ft.
2 in.	20 ft.	6 in.	60 ft.
3 in.	30 ft.	1 ft.	120 ft.

Cost of Supplying Water* varies in the United States from $20 per million gallons in some large cities to $300 in small, unfavorably situated plants; the average of 22 cities for 1904, none larger than Cleveland, was $92, Cleveland being $23. Chicago is estimated at $19.—(E. F. Cole, Technology Quarterly, June, 1907.)

* Total maintenance plus interest on bonds.

Water Charges—Meter Rates in Several Cities

Cents per Thousand Gallons

(From pamphlet of Bureau of Water, Philadelphia, 1914)

Boston	10c. to 18c.	Washington	5c.
Pittsburgh	10c. to 18c.	Montreal	15c.
Baltimore	8c. to 10c.	New York	13c.†
Cincinnati	10c.	Yonkers	20c.‡
Jersey City	10c. to 16c.	Philadelphia (present)	4c.
Rochester	10c. to 14c.	Philadelphia (suggested)	12c. (domestic)
Providence	10c. to 20c.		8c. (intermediate)
Cleveland	5c.		4c. (manufacturing)

Truck Loads on Valve Chamber Covers. Fifty-ton girders on a truck weighing 16 tons (one of the largest in New York City) broke through 24-in. by 24-in. cast-iron manhole covers on Broadway (Mar., 1912) presumably due to impact. The major part of the load was on the rear axle and was computed to be 22 tons per wheel. A 68-ton girder for the Pennsylvania terminal was transported through city streets; also the 70-ton girders for the Woolworth building. Sometimes it is cheaper to ignore such abnormal loads, and replace broken covers. An 80-ton girder gives a wheel-load of about 29 tons. The 24-in. by 24-in. covers of the Metropolitan Street Railway Co. mentioned above were capable of sustaining a static load of 30 tons.

Tar coating for concrete surfaces to prevent erosion by water at high velocities was used in the regulating outlets of Arrowrock dam by Construction Engineer, C. H. Paul. Diam. of outlets 4 ft. 4 in.; surface 1:2 Portland cement mortar; velocities of water, 60 ft. To fill all minute voids, surfaces were scraped and washed with grout then painted two coats of water-gas tar and two coats of coal tar. It is important that the water-gas tar be of very thin consistence; it can be so obtained if so ordered of the manufacturers, who can also supply an oil for thinning, if necessary; success depends upon having this tar of a waterlike consistence. Both water-gas and coal tars should be refined; they may be obtained from Barrett Mfg. Co. Concrete should be thoroughly dry when first coat is applied, but water-gas may be applied without heating; the second coat may follow the first immediately. Both coats of coal tar should be applied hot and brushed out thin as possible; first may be applied as soon as the water-gas tar has soaked in a little, but second should not be put on until first has set. A thick coating is likely to peel and run. After a year's trial, these coatings were found to have been thoroughly satisfactory. The cost was about 1¾ cents. per sq. ft. For 1000 sq. ft. there were required: labor, 4½ days at $2.50; 12 gal. water-gas tar and 15 gal. coal tar, at 16 cents; brushes and miscellaneous, $1.43. Price of tar was $4 per 50-gal. barrel plus freight $4.*

* Reclamation Record, Jan., 1916; E N Jan 16, 1916.
† 10c. per 100 cu. ft. ‡ 15c. per 100 cu. ft.

THREE-PLACE LOGARITHMIC TABLES*

Table 217a. Logarithms of Numbers

N	Log	N.	Log	N	Log	N.	Log	N.	Log	N.	Log
20	301	50	699	80	903	110	041	140	146	170	230
21	322	51	708	81	908	111	045	141	149	171	233
22	342	52	716	82	914	112	049	142	152	172	236
23	362	53	724	83	919	113	053	143	155	173	238
24	380	54	732	84	924	114	057	144	158	174	241
25	398	55	740	85	929	115	061	145	161	175	243
26	415	56	748	86	934	116	064	146	164	176	246
27	431	57	756	87	940	117	068	147	167	177	248
28	447	58	763	88	944	118	072	148	170	178	250
29	462	59	771	89	949	119	076	149	173	179	253
30	477	60	778	90	954	120	079	150	176	180	255
31	491	61	785	91	959	121	083	151	179	181	258
32	505	62	792	92	964	122	086	152	182	182	260
33	519	63	799	93	968	123	090	153	185	183	262
34	531	64	806	94	973	124	093	154	188	184	265
35	544	65	813	95	978	125	097	155	190	183	267
36	556	66	820	96	982	126	100	156	193	186	270
37	568	67	826	97	987	127	104	157	196	187	272
38	580	68	833	98	991	128	107	158	199	188	274
39	591	69	839	99	996	129	111	159	201	189	276
40	602	70	845	100	000	130	114	160	204	190	279
41	613	71	851	101	004	131	117	161	207	191	281
42	623	72	857	102	009	132	121	162	210	192	283
43	633	73	863	103	013	133	124	163	212	193	286
44	643	74	869	104	017	134	127	164	215	194	288
45	653	75	875	105	021	135	130	165	217	195	290
46	663	76	881	106	025	136	134	166	220	196	292
47	672	77	886	107	029	137	137	167	223	197	294
48	681	78	892	108	033	138	140	168	225	198	297
49	690	79	898	109	037	139	143	169	228	199	299
50	699	80	903	110	041	140	146	170	230	200	301

* John Wiley & Sons, 1900.

PART V
CHARACTER AND TREATMENT OF WATER

CHAPTER XXX

CHARACTER OF WATER

GENERAL PROPERTIES

Properties of Pure Water. Pure water is composed of 2 atoms of hydrogen and 1 of oxygen (H_2O). It is a liquid having a blue color and it dissolves most substances, even rocks and metals, to a greater or lesser degree. It is nearly incompressible. Freezing point: 0° C. (32° F.) Boiling point: 100° C. (212° F.)

Table 218. Densities and Volumes of Water from 0° to 100° C.

(At atmospheric pressure = 760 mm. mercury)

Unity at 4° C. Chemiker Kalender

$t°$ C.	Weight of 1 c.c. water in grams	Volume of 1 g water in c.c	$t°$ C	Weight of 1 c c water in grams	Volume of 1 g. water in c c.	$t°$ C	Weight of 1 c c water in grams	Volume of 1 g water in c c.
0	0 999878	1 000122	14	0 999297	1 000703	40	0 99236	1.00770
1	0 999933	1 000067	15	0 999154	1 000847	45	0 99035	1 00974
2	0 999972	1 000028	16	0 999004	1 000997	50	0 98817	1 01197
3	0 999993	1 000007	17	0 998839	1 001162	55	0 98584	1 01436
4	1 000000	1 000000	18	0 998663	1 001339	60	0 98334	1 01694
5	0 999992	1 000008	19	0 998475	1.001527	65	0 98071	1 01967
6	0 999969	1 000031	20	0 998272	1 001731	70	0 97789	1.02261
7	0 999933	1 000067	21	0 998065	1 001939	75	0 97493	1 02570
8	0 999882	1 000118	22	0 997849	1 002156	80	0 97190	1.02891
9	0 999819	1.000181	23	0 997623	1 002383	85	0 96876	1.03225
10	0 999739	1 000261	24	0 997386	1 002621	90	0 96549	1 03574
11	0 999650	1 000350	25	0.997140	1 002868	95	0 96208	1 03941
12	0 999544	1 000456	30	0 99577	1 00425	100	0 95856	1 04323
13	0 999430	1 000570	35	0 99417	1 00586			

g = gram.

Table 219. Solubility of Various Gases, C.c. per Liter, in Pure Water, at Different Temperatures and Normal Pressure (760 Mm.)

$t°$ C.	$t°$ F.	Carbon dioxide (Bunsen)	Oxygen (Winkler)	Nitrogen (Winkler)	Hydrogen sulphide (Bunsen)	Atmospheric air (Bunsen)
0.0	32.0	1796.7	48.90	23.48	4370.6	24.71
5.0	41.0	1449.7	42.86	20.81	3965.2	21 79
10.0	50.0	1184.7	38.02	18.57	3855.8	19.53
15.0	59.0	1002.0	34 15	16.82	3232.6	17.95
20.0	68.0	901.4	31 02	15 42	2905.3	17 01
25.0	77.0	— — — — —	28.31	14.32	2609.1	14.02
30.0	86.0	— — — — —	26 08	13.40	2329 0	12.75

At 20° C. water dissolves 35 c c. methane per liter

Table 220. Weight of 1 Liter of Various Gases at 0° C. and 760 Mm. Pressure

Nature of gas	Formula	Weight, grams	Specific gravity, air = 1
Carbon dioxide	CO_2	1.966	1.519
Nitrogen	N_2	1.255	0.970
Oxygen	O_2	1.430	1.105
Hydrogen sulphide	H_2S	1.523	0.656
Air	....	1.294	1 000
Hydrogen	H_2	0 0896	0 0695

Conversion of c.c. per liter to milligrams per liter may be effected by the following formula:

$$\frac{V}{2W} = \text{milligrams per liter, when}$$

V = c.c. per liter (Table 219)

W = weight of 1 liter of the gas in grams (Table 220)

2 = molecular factor for bivalent gases; all gases in Tables 219 and 220 are bivalent.

Absorption of Gases from Air by Water. On account of partial pressure being exerted, water does not absorb as much of any one gas from the air as when pure gas is brought in contact with water. The volumes of oxygen absorbed from the air by water at different temperatures are given in Table 219. The volume of any gas dissolved by water at constant temperature varies directly with the pressure.

Absorption of Light by Water.—Dr. Birge states that at a depth of 1 meter the solar energy varies in different lakes from 2 to 20 per cent. of that at the surface. That the intensity of light does not decrease geometrically as the depth increases arithmetically, is chiefly because the water is blue in color and absorbs red and yellow rays most readily.

CLASSES OF WATER

Meteoric Water. All water derived directly from the atmosphere is called meteoric water, whether in the form of dew, rain, snow, sleet or hail.

Along the Gulf shores of U. S., where the average annual rainfall approaches or exceeds 60 in., rain-water supplies are the rule in all but the largest cities; until 1903 even New Orleans depended largely upon rain water stored in wooden cisterns for its drinking water supply. Water evaporated from fresh-water lakes and condensed on high mountains is exceptionally pure, but rain and snow collected in towns is more or less polluted with the washings of air and roofs, especially during the beginnings of storms. Examples of rain and snow water are given in Table 243, page 670 (Samples 69 to 75). Cohen and Ruston, in "Smoke, a Study of Town Air," 1912, determined the total impurities washed from the air at Leeds (Eng.) Experimental Farm 6 miles from the city to be as follows:

Substance		Tons per square mile per annum
Insoluble.....	Carbon........................	87.1
	Tar	42 6
	Ash........................	262.5
	Total...........	332.2
Sulfur as.....	Free acid	8.5
	Sulphates	77 0
	Other compounds	19.1
Nitrogen as...	Free ammonia.	4 1
	Albuminoid ammonia	1 0
	Nitrate	0 14
	Total	5 24

Chlorine in Rain Water. Along the coast, rain water contains small amounts of sodium chloride borne from the sea, the amount varying with the distance from the sea and the direction of the air currents. See chlorine map, Fig. 411, page 800.

Collection and Purification of Rain Water. Rain water should be collected from clean surfaces and the first runnings from the collecting surface should be rejected. It should be stored in wooden or masonry cisterns, never in lead-lined or unprotected steel tanks. Cistern overflows should not connect with sewers. It is usually necessary, especially in towns, to filter the portion collected in order to make it satisfactory. Rain water usually has a mawkish taste which may be overcome by filtration and aeration. Rain-water filters should be designed with care; water should be stored before filtration to insure a low rate of filtration. For supplying a residence at Little Compton, R. I., having a catchment area equivalent to 2500 sq. ft., in the form of a shingled roof, with 400 gals. daily for 4 months yearly, there were required a rain-water basin holding 10,000 gals., to utilize the run-off during a heavy shower, a slow sand filter 9 sq. ft. in area, and a filtered-water basin holding 40,000 gals. Cost, $1500, exclusive of pumps, motors and piping. Rain-water filters should have at least a 3-ft. depth of sand having an effective size of from 0.15 to 0.30 mm. Maximum rate of filtration, 6 mgad. Filters should be examined frequently and scraped when necessary, and provision should be made for aerating the water filtered. Accumulated sediment in cisterns should be removed frequently.

Gases in Rain Water. Water dissolves from air relatively larger proportions of carbon dioxide than of oxygen, and of oxygen relatively larger proportions than of nitrogen, notwithstanding the ratio of $N : O$ in the air $= 4 : 1$. One liter of water, under 760 mm. pressure and at the following temperatures, dissolves the following volumes of gases from the air. (Fischer, "Handbuch der Chem. Technologie," 1893.)

Gas	Temperature, ° C.		
	0	10	20
Oxygen..........	41 cc.	33 cc.	28 cc.
Nitrogen..	20 cc.	16 cc.	14 cc.
Carbon dioxide..	1797 cc.	1185 cc.	901 cc.

Rain water contains traces of the omnipresent ammonia and nitrates, and occasionally traces of nitrites. The amount of nitrogen as free ammonia varies between 0.2 and 5.0 p.p.m., and as nitrate from 0.2 to 2 parts per million. (Table 243, p. 670.) Rain water contains ozone (O_3) which is not a purifying agent, per se. The so-called ozone process of water purification depends upon the production of traces of hydrogen peroxide (H_2O_2). (*Blücher*, "Das Wasser," 1900.) Angus Smith found 35 p.p.m. of sulphuric acid in rain water in Liverpool, 50 parts in Manchester, 430 parts in Newcastle and 20 parts in London. In the vicinity of smelters and chemical works, rain water may contain hydrochloric, nitric and other acids, nitrous oxide, carbon monoxide, and oxides of zinc, lead, copper and other metals.

Ground Water. The meteoric water which sinks into the ground and flows seaward along more or less impenetrable strata, is called ground water—subdivided for convenience into spring, shallow well, deep well and artesian water.

Appearance. Meteoric water falling upon the earth and passing through thick enough layers of proper soil loses its suspended matter and becomes clear. An exception is found where fine particles of clay are borne with the water through the relatively large pores of water-bearing soil or the fissures and seams in the rock. Waters which become turbid directly after rains are usually polluted from surface sources. Some waters which are clear when drawn become *turbid* on standing, due to the precipitation of iron, manganese, calcium and magnesium carbonates, etc., generally following the escape of the gases which hold these substances in solution. Ground waters are usually *colorless*, but not always so. The deep well waters of the Mississippi delta often have a color of 1400 and waters from near peat deposits are also colored. The color is due to humus bodies of obscure composition (humic acid and ulmic acid). These are frequently associated with iron and manganese (iron humate, etc.).

Odor. Most ground waters have little or no odor. Some, however, are impregnated with hydrogen sulfide or sulfur dioxide and possess the characteristic odors of these gases. Polluted wells may have a musty or disagreeable odor, while ground waters from marshy sources often give off the characteristic odor of peat, especially when heated.

Tastes of ground waters are usually imparted by dissolved gases and minerals. The agreeable taste of carbonic acid and the disagreeable taste of hydrogen sulfide are well known; waters containing iron or manganese are characterized by a styptic or "inky" taste; the hardness of water is often apparent to the taste, especially when large amounts of magnesium are present. Some waters contain so much sodium chloride that it can be detected by the taste. The minimum amount of sodium chloride which can be so detected ranges, according to various observers, from 200 to 300 p.p.m. Few observers can detect 200 p.p.m. or less of sodium chloride, even when dissolved in distilled water.

Temperature. One valuable property of ground waters is the small annual range in temperature. The temperature of shallow well waters is rarely over 20° C., and rarely lower than 6° C. Waters from greater depths are practically constant in temperature. E. A. Martel ("Comptes Rendus," Mar. 13,

1913) states: "A constancy of temperature exists only in the continuous flows from sand and similar material or artesian supplies from great depths, slowly and regularly maintained." Ground waters are cooler in summer and warmer in winter than surface waters, therefore less liable to cause freezing of services in winter and more agreeable for drinking purposes in summer.

Organic Matter in the soil and that passing through it with the ground water is of two main kinds, carbonaceous and nitrogenous. Theoretically these may become oxidized to CO_2, CH_4 and H_2. These end products are rarely reached in nature as the result of decomposition of organic matter in the soil. Carbonaceous matter comprises the cellulose, lignin and chemically similar bodies; the nitrogenous matter comprises the albumins, the waste of animal life and proteids. Carbonaceous matter may be oxidized to carbon dioxide, and the nitrogenous matter may be oxidized to nitrates. Rideal

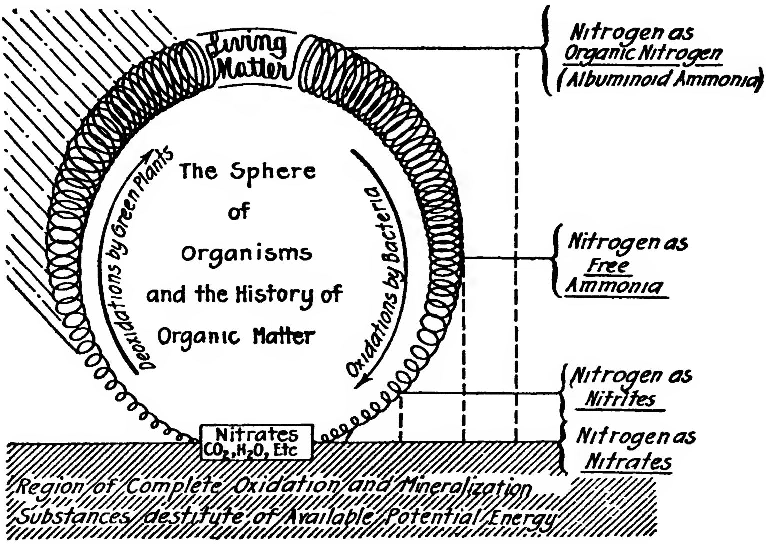


FIG. 343.

("Water Supplies," 1915) divides nitrogenous decomposition into three periods: firstly, the formation of poly-peptides from proteids, a period of which sewage is a typical example; secondly, the production of amino-acids; lastly, the conversion into ammonium compounds, which are oxidized into nitrites and nitrates. Organic matter and the bacteria which decompose it, are most abundant in the surface layer of the soil, where also there is the largest amount of available oxygen.

The circulation of nitrogen from the organic to the mineral state is known as nitrification, and is brought about by the nitrifying bacteria (discovered by Winogradsky). These bacteria change ammoniacal nitrogen to nitrites and nitrates. Nitrates are readily taken up by green plants through the constructive power of chlorophyll. When the living matter again breaks down, free ammonia and nitrates and nitrites are formed as before. This

cycle is illustrated in Fig. 343 devised by W. T. Sedgwick (1889) to illustrate the transformations of nitrogen in nature in a never-ending cycle, through the vitalized and mineralized stages in turn. Certain bacteria, called denitrifying, reduce nitrates with the production of nitrites. Some bacteria, in turn, reduce nitrites to ammonia and ammonia to nitrogen, thus reversing the ordinary course of the nitrogen cycle.

Dissolved Gases. Carbon dioxide which results from decomposition of organic matter is the gas of first importance in ground waters, which contain little or no oxygen. Meteoric waters contain little carbon dioxide but are nearly saturated with oxygen. As meteoric water passes into the soil, its oxygen is used to decompose organic matter contained in the soil and water. Frequently deep wells contain extraordinary amounts of nitrogen in the form of free ammonia, particularly where there are fossil, animal or vegetable deposits in the water-bearing strata. Bartow (Report of Illinois State Water Survey, 1907–1910) reports several cases illustrating this phenomenon. Such waters from sources of unquestioned purity, may contain even more than 2.0 p.p.m. of nitrogen as free ammonia.

Effect of Carbon Dioxide. Production and absorption of CO_2 greatly increase the solvent action of water. Lime, magnesia, iron, manganese and other elements are rendered soluble, while the silicates and other minerals are decomposed, setting free the soluble elements composing them. Ground waters, therefore, contain more substances in solution but less in suspension than do surface waters. Humic or other organic acids may exert a similar effect on the soil. If dissolved oxygen be absent from the ground water, sulfates may be reduced to sulfides and hydrogen sulfide set free. Similarly nitrogenous organic matter is reduced, with the production of ammonia and nitrogen.

Oxygen Requirement of Soil. Because the purifying action in the soil requires oxygen, the discharge of contaminating matter on the surface in the vicinity of a ground-water supply must be intermittent if the purity of the ground water be maintained. This is the condition which occurs in nature, a period of aeration following each period of rainfall. In cities and towns, pavements and buildings prevent aeration of the soil and consequently the purification of the water thereby. Sometimes purification is disturbed or arrested to such a degree that even wide zones of soil of the right physical character fail to protect ground-water sources against contamination.

Efficiency of Natural Purification. The distance which a water must pass through the soil before becoming safe is important, and depends upon local conditions, such as rate of flow, character of soil, and degree of pollution. Again the work imposed upon the soil may exceed its capacity; aerobic bacteria may be deprived of oxygen and give way to anaerobic bacteria, a condition not so favorable to bacteriological purification of ground water. A well-aerated soil is much more efficient than a soil which has been robbed of its oxygen.

Before sinking wells at Wuhlheide and Heiligensee, Ditthorn and Luerssen (Gesundheits-Ingenieur, 1909) made a careful study of the permeability of the soil to bacteria. The object of the test was to ascertain how near a well could be to a broken sewer pipe without danger of contamination. They used B.

prodigiosus for their experiments. The first of these series of experiments was made at Tegel See where one of the waterworks wells, which was driven to a depth of 177 ft., and whose strainers began at a depth of 121 ft., was set apart for this purpose. At a distance of 69 ft. from this well, a pipe 62 ft. long was driven into the ground and through it the bacteria were introduced into the soil below ground-water level through a strainer on the bottom of the pipe. At this point the ground-water level was but a few meters below the surface. A particular distance, 69 ft., was chosen because some of the proposed wells were to be that distance from a sewage line. The soil was made up of gravel and sand a little finer on the average than good mortar sand. Bacteria were introduced into the side pipe and water was pumped in afterward, until the level was 3 to 4.5 ft. above ground-water level, forcing the water containing the bacteria into this soil and directly into the ground-water stream. The test was conducted for several days continuously and 380,000 gals. of water were pumped from the well daily. During the first 45 days, samples were taken from the well daily. During the first 11 days of the test, 61,000,000 bacteria were put into the side pipe. Bacteria were detected in the water drawn from the well on 10 different days—the first time 9 days after the first injection of bacteria into the soil, the last time 19 days after the last injection. There were two intervening periods, one of 5 and one of 6 days, during which no bacteria were found. About one bacterium in 40,000, according to computations, reached the well water. It was concluded that the dangerous bacteria are much more sensitive and die out sooner than B. prodigiosus, and therefore the distance between the well and side pipe afforded sufficient protection.

In a second series of experiments made at Müggel See, bacteria were introduced above the ground-water level. The well was 140 ft. deep; the top of the strainer was 92 ft. below the surface and about 76 ft. below ground-water level, which in turn was about 16 ft. below the surface. The device for introducing the bacteria into the soil was an imitation broken drain. This was located 58 ft. from the well and was 7 ft. below the surface, therefore 9 ft. above ground-water level. The soil was a mixture of coarse and fine sand. The tests lasted 1½ months. Pumps drew an average of 126 gals. per hour from the well. At the same time, 700 gals. of water per hour, on an average, were forced into the germ pipe. No positive results were obtained; then the germ pipe was lowered until it was only 4 ft. above ground-water level, and the quantity pumped from the well was increased to 3540 gals. per hour. In no case, however, were bacteria found in the well water. This test supports the general opinion that a wet zone is never so sufficient a protection against pollution as a dry zone.

Bartow (Report of Illinois State Water Survey, 1907–10) made a careful study of the quality of Illinois ground waters. There are three main classes of water in Illinois, namely, from deep wells, from wells in glacial drift, and from wells in the clay-like loess. The last source has failed due to lowering of the ground water as the result of drainage for agricultural purposes. Safety of Illinois ground water increases with depth below surface.

Sometimes a zone of soil 10 ft. wide will prevent the passage of contaminat-

ing material to a well, but ordinarily a greater width is essential; 50 ft. of fine sand nearly always suffices, provided ground-water level is 10 or more feet below the surface. Where faults and fissures abound, as in limestone formations, contamination may pass a long distance through soil. At Lausanne, Switzerland (Deutsch arch. f. klin. Med., 1893, II), the village well was polluted from a stream more than a mile away. Gaffky (Mit a. d. kais. Gesundheitsamt, 1884, II, p. 413) reported the infection of an open well from a privy vault 50 ft. distant.

Bacteria in Ground Water. Numbers of bacteria in soil and in ground water decrease rapidly with depth. Kabrhel (Archiv für Hygiene, 58, 345, 1906) found several million bacteria per c.c. in surface samples of woodland soil, a few thousands 0.5 meter below the surface, and usually only hundreds per c.c. in samples collected from depths greater than 1 meter. Many organisms in ground water grow slowly and therefore do not appear after short periods of incubation, thus giving rise to the erroneous belief that ground waters from considerable depths are invariably bacteria free. S. C. Prescott examined 147 shallow farmyard wells and found that 124 contained no B. coli and were therefore probably free from fecal pollution. These samples averaged 190 bacteria per c.c., while the 23 samples which gave positive tests for B. coli averaged 570 per c.c. Distribution of the two series of samples according to the number of bacteria present is indicated in Table 221.

Table 221. Bacteria in Shallow Farmyard Wells

Percentage of Samples in Each Group

Bacteria per c.c.	0	1–10	11–20	21–50	51–100	101–500	501–1000	1001–2000	2001–3000
Series I. B. coli absent	3	16	14	16	11	31	5	4	
Series II. B. coli present			5		10	57	10	14	5

Kellerman & Whittaker (U. S. Dept. of Agri., Bureau of Plant Industry, Bulletin 154, 1909) report similar results for *shallow* wells used as farm water supplies. Egger (Wolffhügel, 1886) examined 60 wells in Mainz and found that 17 of them contained over 200 bacteria per c.c. Maschek (1887) found 36 wells out of 48 examined in Leitmeritz which had a bacterial content of over 500 per c.c. Fischer (Horrocks, 1901) reported 120 wells in Kiel which gave over 500 bacteria per c.c. and only 51 with less.

Deep well waters contain very few bacteria. Houston (32d Annual Rep., Local Govt. Bd. containing Rept. of Medical Officer for 1902–03, 581) found the bacteria in the waters from a series of deep wells of high quality at Tunbridge Wells, England, to vary from 1 to 36 per c.c. Prescott (Elements of Water Bacteriology, Prescott & Winslow, 1913, p. 27) found in the water of 15 driven wells in the vicinity of Boston, examined in 1903, an average of 18 colonies per c.c. at the end of 48 hours' incubation. Prescott & Winslow (Elements of Water Bacteriology, 1913) found the following numbers of bacteria in certain wells and springs from various sources:

Table 222. Bacteria in Deep Well and Spring Waters

Town	Bacteria per c c.	Town	Bacteria per c.c.
Worcester, Mass.	10	Saranac Lake, N. Y.	11
Waltham, Mass.	3	Ellenville, N. Y.	0
Newport, R. I.	7	Hyde Park, Mass.	12

Fuller (Jour. Franklin Inst., July, 1915) quotes results of "an instructive series of investigations recently made by Dr. Ernst Quantz, at Göttingen, Germany (*Zeitschrift für Hygiene*, 1914, vol. 78, p. 193), see Table 223. Group I consists of wells which are practically impossible of pollution, and may be considered as safely good. Group II are doubtful, and at times might be polluted and at other times be safe. Group III are particularly bad wells, and nearly certain to be polluted. In examining these data a general relationship

Table 223. Bacterial Count of Well Waters

Bacterial count per cubic centimeter	Number of examinations			Per cent. of examined wells		
	Group I	Group II	Group III	Group I. Good	Group II. Doubtful	Group III. Bad
0— 10	2	0	0	6	0	0
11— 50	5	10	0	16	9	0
51— 100	3	10	0	9	9	0
101— 200	6	12	1	19	10	3
201— 500	6	22	4	19	19	11
501— 1000	6	18	7	19	15	20
1001— 2000	3	18	7	9	15	20
2001— 5000	1	13	5	3	11	14
5001—10000	0	11	3	0	9	9
10001—20000	0	1	5	0	1	14
Over—20000	0	2	3	0	2	9

is shown. Thus, taking 1000 bacteria per c.c. as the limit, in Group I, 88 per cent. are under this limit; in Group II, 62 per cent.; and in Group III, 34 per cent. Taking 100 bacteria as the limit, in Group I, 31 per cent. are under this limit; in Group II, 18 per cent.; and in Group III, none. The bacterial count was made on gelatine at 20° C. This indicates that the better wells, on an average, have a lower bacterial count than the poorer wells. Individual wells, however, show no such relationship. In some cases good wells show a higher bacterial count than poor wells. Often the rate of draft is a bigger factor in the number of bacteria shown than the possibility of pollution. Good wells which have been standing for some time show a very high bacterial count.

Bacillus Coli in Ground Waters. B. coli, which are normal inhabitants of the human intestine and of intestines of various animals, are found in polluted ground waters. B. coli also exist outside the bodies of animals, and organisms giving the reactions of B. coli may live in a semi-parasitic fashion on plants as well as on animals. (Prescott, Science N. S., **15**, 363; also Medicine, **11**, 1912, 20, and "Biological Studies by the pupils of W. T. Sedgwick," 1906.) The term B. coli covers a number of forms or types; it really represents a whole class of bacteria. Therefore it is the number found in a ground water, as in a surface water, rather than their mere presence, which is of sanitary importance. B.

coli results should be used most cautiously in judging the quality of a water, largely because the "presumptive test" for these bacteria tends more and more to include organisms of the coli group as well as the typical bacillus of the human intestine. B. coli may occasionally be found in ground waters of good quality, provided large enough volumes be tested. The bacilli abound at the ground surface and a very few may penetrate to great depths. That their mere presence proves the presence of infinitely rarer disease germs is inconceivable.

Mass. State Bd. of Health (Report 1901) examined samples from 99 springs and found B. coli in 6. All were liable to pollution. The same investigation showed that bottled spring waters were subject to contamination during bottling, B. coli being found in 7 samples where none were found in the corresponding samples from the springs. Clark and Gage (Rept. Mass. State Bd. of Health, 1903) found B. coli in 5 out of 170 samples from tube and shallow wells of good construction. Houston (loc. cit.) compares the more or less polluted shallow wells of Chichester with the excellent deep ground waters from Tunbridge Wells. Table 225 (Prescott and Winslow, p. 162) shows the value of 1 c.c. samples in differentiating between good and bad waters.

Table 225. Distribution of B. Coli in Good and Bad Well Waters

(HOUSTON, 1903)

Percentage of Positive Tests

Quantity of water	Chichester, shallow wells	Tunbridge Wells, deep wells
100 c.c.	90	25
10 c.c.	80	6
1 c.c.	45	0
0 1 c.c.	20	0

Fromme (W. Zeit. für Hygiene, 65, 251) shows relation between numbers of bacteria and B. coli as found by examining 120 samples from wells near Hamburg, Table 226.

Table 226. Relation between Total Numbers of Bacteria and B. Coli

(FROMME, 1910)

Colony count	Number of samples	Per cent positive; B. coli tests in 10 c.c.
Over 200	35	40 0
50–200	19	15 8
Under 50	66	3 0

The permissible number of B. coli in a ground water is difficult to determine. B. coli should not be present in 1 c.c. samples nor in more than 20 per cent. of 10 c.c. samples. B. coli will rarely be found in the best ground waters, even in 100 c.c. samples, nor in more than 10 per cent. of the 10 c.c. samples from most undeniably safe sources. However, certain exceptional waters from the vicinity of peaty deposits, such as exist on Cape Cod, Long Island and elsewhere, contain bacteria showing most of the B. coli reactions. Such waters should not be judged by the presumptive test* for B. coli alone.

* See page 795.

Mineral Content of Ground Water. According to Tiemann & Gärtner (Handbuch der Wasseruntersuchung, 1895), the substances dissolved in normal ground waters of North Germany lie as a rule within the following limits, which limits exclude strongly impregnated mineral waters:

Substance	Parts per million
Total residue on evaporation	Less than 500
Calcium and magnesium (Ca + Mg)	Less than 150
Chlorine (Cl)	Less than 30
Sulphates (SO_4)	Less than 120
Nitrogen as nitrates	Less than 5
Nitrogen as nitrites	Less than 0.01
Nitrogen as free ammonia	Less than 1
Oxygen consumed (O)	Less than 2.5

Most of the acids and bases present in natural waters are in the combined state. According to Reichardt,* ground waters from various geological formations vary as shown in Table 227.

Table 227. Composition of Ground Waters from Different Geological Formations (Reichardt)

(Parts per Million)

Formation of water-bearing stratum		Residue on evaporation	N_2O_5*	Cl	SO_3	CaO	MgO	Oxygen consumed
Granite	a	24 4		3 3	3 9	9 7	2 5	0 73
	b	70 0		1 2	3 4	30 8	9 1	0 20
	c	210 0		Trace	10 3	44 8	21 0	0 23
Trap		160 0		8 4	17.1	61 6	22 5	0 96
Basalt		150 0		Trace	3 4	31 6	28 0	0 10
Claystone Porphyry		25 0		..	3 4	5 6	1 8	0 41
Clay schist	a	120 0		2 5	24 0	50 4	7 3	0 89
	b	60 0		8 8	1 7	2 8	3 6	0 86
	c	70 0	Trace	2 0	5 0	5 6	1 8	1 03
	d	180 0	Trace	10 6	10 0	44 0	10 8	0 71
Sandstone	a	225 0	9.8	4 2	8 8	73 0	48 0	0 46
	b	300 0	4.0	3 2	3 4	95.2	7 2	0 20
	c	190 0	Trace	8 9	27 5	39.2	28.0	0 13
	d	90 0	.	7 5	.	10 0	3 6	0 35
Chalk		325 0	0 2	3.7	13.7	129 0	29 0	0 28
Dolomite.		418 0	2 3	Trace	34.0	140 0	65 0	0 28
Gypsum...		2365 0	Trace	16.1	1108.3	766 0	122 5	Trace

* Nitrogen pentoxide.

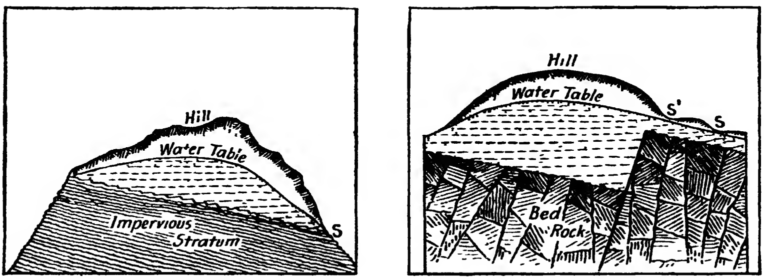
Sanitary analyses of ground waters are given in Table 243 (Samples 76 to 84), p. 670. For results of mineral analyses of other American ground waters, see Reports of U. S. G. S., Water Supply Papers Nos. 6, 12, 21, 78, 101, 111, 114, 123, 136, 137, 138, 157, 159, 160, 164, 182, 183, 190, 191, 195, 199, 223, 225, 227, 232, 254, 258, 259, 319, 333, 335, 338, 341, and particularly No. 364 (1897–1914).

Ground Water Zones. Underground waters are divided by Slichter into 3 principal zones: The unsaturated zone; the surface zone of flow; and the deeper zone of flow. Motion of water in the unsaturated zone is essentially vertical, downward after a fall of rain and upward when supplying surface vegetation by capillary action. The surface zone of flow extends from the

* "Grundlagen zur Beuteilung des Trinkwassers," E. Reichardt, Halle, 1880.

level of the water table to the first impervious stratum. The deeper zones of flow are those which lie below the first impervious stratum. Strata may be impervious enough to collect a layer of ground water, yet permeable enough to cause the formation of the deeper zones of flow. All ground waters have a tendency to move in the direction of the neighboring streams and lakes. Where the slope is great, the underflow in the direction of the thalweg (valley-way) may be relatively very large, but seepage in the steeper slopes and terraces of the river valley is primarily in a direction toward the thalweg and not along it. The underflow of streams is usually very small and its velocity very low. Most streams deposit enough silt to make their banks and bottoms practically impervious.

Spring waters, which may be of high purity before emergence at the surface, may become polluted by surface drainage because of the conditions which surround the spring. In many cases such springs can be "restored" by following the rules given below for shallow wells, and by replacing the soil about the spring for a depth of from 3 to 5 ft. or more, and for a width of 25 ft. or more, with a zone of fine sand. Fig. 345 illustrates this method of protection.



Stratum Spring   Fault Spring

FIG. 344.

Protection of Shallow Wells. Shallow wells are those sunk by digging or driving to depths of less than 30 ft. Because the depth of the ground-water level below the surface is the most important single factor from a hygienic standpoint, shallow and deep wells are considered separately. One distinguishes between dug and driven, or tube, wells in this connection because of the greater safety of the latter, the depth of the ground-water level being equal. Nevertheless, if great care be taken, dug wells can be made nearly if not quite as safe as tube wells. In order to protect dug wells, the following rules should be observed:

1. Lining of well should be tight from surface of ground to water-bearing stratum; for a depth of 5 ft. in any case, better 10 ft.

2. Lining of well should be brought at least 1 ft. above surface.

3. Well should have a tight cover and the ground immediately around is should be raised so that surface drainage will be away from the well rather than toward it. Preferably the cover of the well should be of concrete or cast iron, and the ground about it should be paved with concrete or other impervious material. (See also p. 240.)

4. Pump should be so arranged that waste water cannot return to well without passing through a layer of soil sufficient to protect the well against contamination.

5. There should be a zone of protection about each well in order to guard it against contamination from accidental polluting discharges in the vicinity of the well. This zone should have a width of at least 5 ft., preferably more.

6. Ventilation openings are usually not necessary. If used, they should be screened to keep out insects and small animals.

So-called "draw wells" with open tops are inferior to wells with pumps, and should be used only in exceptional cases. Wooden covers are likely to leak.

Conditions which surround tube wells are exactly the same as those which surround other wells of the same depth. Their advantage lies in the greater protection afforded by the tight well tube. The tubes themselves, however, dissolve rapidly in soft waters containing large amounts of free CO_2, thereby giving rise to leaks and making renewals frequent and expensive.

Deep wells differ from shallow wells chiefly in their hydrology. They are ess often polluted than shallow wells, but are apt to contain more dis-

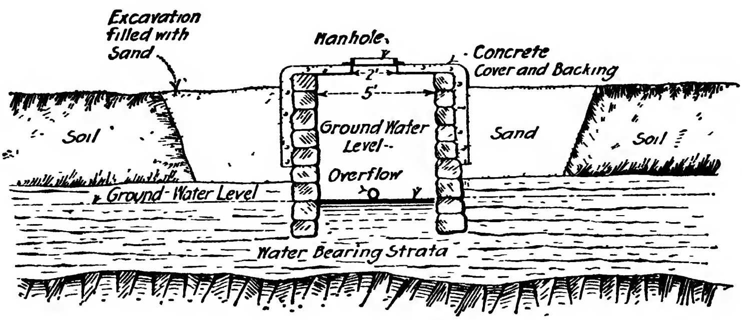


FIG. 345.

solved mineral matter. Sometimes deep wells are affected by fossil or peaty deposits.

Artesian wells are wells, shallow or deep, in which the water-level is raised above the normal ground-water level by subterranean pressure.

Infiltration galleries, or horizontal wells. Many rivers carry so much silt that their banks and bottoms are practically impervious. Consequently infiltration galleries or wells in their vicinity draw little water from the surface stream, but chiefly from the ground-water stream in the river valley. Certain river valleys, however, are composed of such coarse material, particularly in mountainous regions, that wells or filter galleries may be used to derive supplies from them. Denver, Colorado, derives a part of its supply from such a source, as do numerous German cities on the Ruhr, Elbe and Rhine rivers. Such supplies, owing to the extreme porosity of the water-bearing layer and the lack of an impervious protecting layer above, are subject to contamination during flood seasons. Many so-called "filter cribs" constructed in gravel beds beneath rivers draw their supply almost exclusively from the river.

Many inclined tube wells driven beneath the beds of silt-bearing rivers fail to draw any considerable part of the supply from the streams themselves. Local conditions determine the source in any case. (See also p. 256.)

Irrigation. The production of "artificial ground water" by irrigating the surface soil in the vicinity of wells with water drawn from a near lying stream, has given good results in many cases, notably at Brookline, Mass., Chemnitz, Germany, and Stockholm, Sweden. It is not to be recommended where the rivers are highly polluted and where the irrigated area is very porous. Its

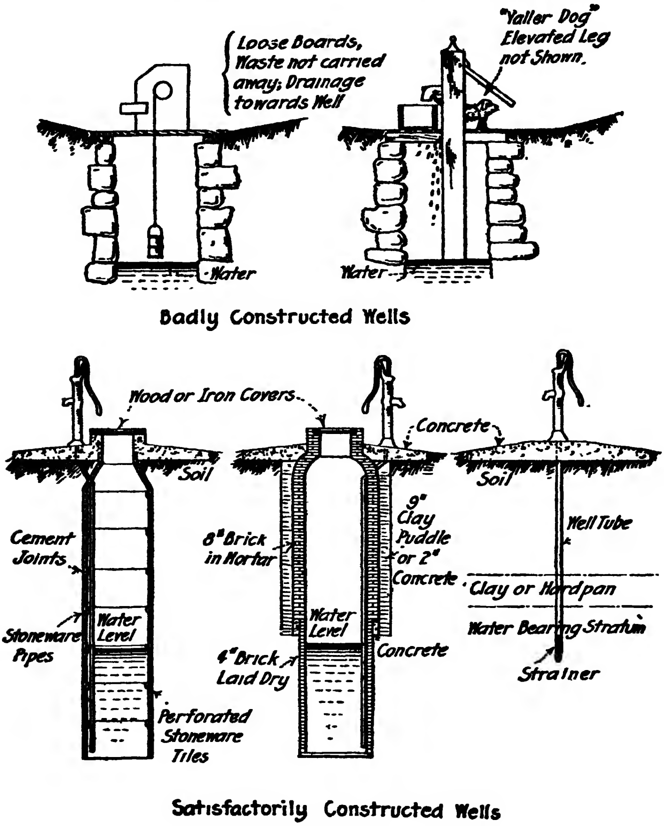


FIG. 346.

efficiency depends largely upon the intermittent application of the river water to the surface sand or silt and upon the construction and operation of the irrigated area. This latter should be readily accessible for cleaning and should be so large that the rate of filtration will not greatly exceed the natural rate of filtration through the soil. Because it can be controlled better, artificial ground water is to be preferred to that drawn through the soil directly from a polluted river.

Surface waters from streams, rivers, ponds, and lakes, the most generally available sources, are especially liable to pollution, and for this reason are

rarely suitable for domestic use without purification, by storage (natural or artificial), treatment with chemicals, filtration or disinfection. Analyses of various surface waters given in Table 243 (Samples 1 to 68), p. 668, show better than a description their varying characters. They differ greatly in suspended matter, including microscopic organisms and bacteria. They have temperatures largely dependent upon climate and season and a varying gaseous content which, in the case of pure waters, is characterized by low CO_2 and high O and N. Certain waters may be supersaturated with oxygen produced by active growth of algæ. Other characteristics of surface water are the generally higher amounts of nitrogenous and carbonaceous organic matter, as shown by higher "loss on ignition," "albuminoid ammonia," color, and "oxygen consumed." Some ground waters which contain humus matter exhibit these characteristics. Surface waters contain less dissolved mineral matter (Si, Ca, Mg, Na and K) and practically no dissolved iron or manganese except that which is in combination with the organic coloring matter.

Dissolved Gases in Surface Waters. Gases expelled by boiling five typical waters were determined by Sir E. Frankland, with results given in Table 228. They included rain water and water from a deep well in the chalk deposit, as well as from three surface sources. The high CO_2 content in the Thames and chalk well waters is due to oxidation of organic matter. The values given include not only the free CO_2 but that in the form of bicarbonates which also is driven off by boiling. Thresh ("Examination of Waters and Water Supplies," 1904, p. 293) found 99.16 per cent. of N, and 0.84 per cent. of CO_2 in gases evolved from Buxton thermal spring, in England.

Table 228. Gases Expelled by Boiling Typical Waters, c.c. per L.
(FRANKLAND)

Kind of gas	Rain water	Cumberland mountain water	Loch Katrine water	Thames water	Deep chalk well water
Nitrogen.	13 08	14 24	17 31	13.25	19 44
Oxygen . . .	6.37	7.26	7 04	5.88	0 28
Carbonic acid	1 28	2.81	1 13	40 21	55 20
	20.73	24.31	25.48	59.34	74 92

Organic Matter in Surface Water. Besides the dissolved organic matter described on page 658, surface waters contain many kinds of suspended organic matter, both living and dead. This is sometimes called plankton, although this term is usually restricted to living organisms (from the Greek word πληνκτος meaning "that which wanders"). Suspended organic matter includes that organized as microscopic plants and animals, the former including the bacteria, which are true plants although some of them possess the power of locomotion. It also includes wastes of plant and animal life and organic matter discharged with sewage from factories and towns. Growth of organisms in surface water is dependent upon food and light. Nitrates and carbon dioxide form the ideal plant food and give rise to abundant growths of algæ. Bacteria which feed upon or peptonize the suspended matter, and bacteria which min-

eralize the dissolved organic matter, with the algæ, furnish food for growths of Protozoa; these in turn form part of the food for growths of Hydræ and Rotifera, all of which are consumed by fish. Oysters and clams also feed upon diatoms and other microscopic organic matter, living and dead; and the abundance of the higher plant life supported indirectly by the organic matter contained in the water in which it grows is well known. Often the only apparent result of increased organic pollution of a body of water is a corresponding increase in fish, oysters, clams, etc., for example, Illinois river (Forbes and Richardson, 1913) and Genesee river (M. C. Whipple and J. W. M. Bunker, 1912).

Fecal waste from the higher animal forms decomposes into substances which, in the presence of oxygen, pass through the nitrogen cycle shown in Fig. 343, p. 647. Some organic matter present in a water is removed in the form of the adult insects whose larvæ live in the water, for example, Chironomas. Solid masses of organic sludge from the bottoms of streams become gradually liquefied by anaerobic bacteria. These sludge masses also serve as food for a variety of worms and larvæ, but only in the presence of oxygen. Liquefied sludge forms part of the food for bacteria and other organisms. There exists, therefore, in many streams containing undecomposed organic matter, a continuous cycle of changes, resulting ultimately in biological equilibrium and, if no new food material be forthcoming, in rapid decadence of organic life and mineralization of the dead organic matter. In other words, growth of organisms, including bacteria, plays the most important rôle in self-purification of streams.

Much organic matter in surface waters is in the colloidal state, that is, in semi-solution. This matter slowly coagulates and subsides. For example, the coloring matter in the highly colored Dismal Swamp water precipitates on standing, although the water apparently contains no suspended matter when first drawn.

For names and descriptions of genera of organisms, as well as their frequency of occurrence, their relation to the chemical constituents of the water and to sizes and depths of reservoirs, see Whipple's "Microscopy of Drinking

Table 229. Frequency of Occurrence of Groups of Organisms in Massachusetts Surface Waters

Classification	Number of genera				
	Commonly found in large numbers	Occasionally found in large numbers	Commonly found in small numbers	Occasionally observed	Total
Diatomaceæ...	5	4	4	22	35
Chlorophyceæ....	3	8	14	21	46
Cyanophyceæ...	4	3	1	8	16
Fungi and Schizomycetes	1	3	3	5	12
Protozoa....	5	5	11	24	45
Rotifera....	0	0	5	12	17
Crustacea....	0	0	3	4	7
Miscellaneous	0	0	0	10	10
Total....	18	23	41	106	188

Water," 1914. Frequency of occurrence of the various groups in Massachusetts waters is summarized numerically in Table 229. Ten genera are especially troublesome in Massachusetts,* namely: Asterionella, Anabæna, Clathrocystis, Cœlosphærium, Aphanizomenon, Dinobryon, Peridinium, Synura, Uroglena and Glenodinium.

Whipple (Report of New York Board of Water Supply, 1906) made a statistical study of 66 lakes and reservoirs in New England and classified them according to their *index of frequency*, calculated as follows: "It was assumed that when organisms were less than 500 per c.c. they would cause no trouble; between 500 and 1000 per c.c., little trouble; between 1000 and 2000 noticeable

Table 230. Lakes Classified According to Index of Frequency of Organisms
(WHIPPLE)

	Group I	Group II	Group III
Numbers of lakes and reservoirs in the group	28	18	20
Limits of frequency index	0–25	25–50	50–100
Average index of frequency	12	39	78
Organisms per c.c., mean yearly average	362	776	1410
Organisms per c.c., minimum yearly average.	54	441	984
Organisms per c.c., maximum yearly average.	1413	2800	3090
Organisms per c.c., mean average for 4 summer months	414	1023	1965
Organisms per c.c., minimum average for 4 summer months	66	227	985
Organisms per c.c., maximum average for 4 summer months	1058	4588	7659

Group I. Organisms often as high as 1000 per c c. Group II Organisms only occasionally as high as 1000 per c.c. Group III. Organisms ordinarily between 100 and 500 per c.c.

Table 231. Algæ in Connecticut Water Supplies
(WHIPPLE)

City	Lake or reservoir	Index of frequency
Bridgeport	Island Brook Supply	83
New Haven	Dawson Lake	79
Meriden	Merimer Lake	74
New Britain	Shuttle Meadow Lake	55
Middletown	Laurel Brook Reservoir	45
Bridgeport	Pequonnock River Supply	40
Hartford	Reservoir No. 6	29
Norwich	Fairview Reservoir	22
New Haven	Whitney Lake	21
Hartford	Reservoir No. 1	18
Bridgeport	Mill River Supply	18
Hartford	Reservoir No. 3	12
New Haven	Wintergreen Lake	12
New Haven	Saltonstall Lake	0
Hartford	Reservoir No. 2	0
Hartford	Reservoir No. 5	0
New London	Lake Konomoc	0

* For other parts of the U. S. no reports of regular examinations are published.

trouble; between 2000 and 3000, decided trouble, and that above 3000 trouble would be serious. From analyses the per cents of the time when organisms were present within these limits were ascertained. These were then weighted as follows and added together: For numbers between 500 and 1000, one-half the per cent.; for numbers between 1000 and 2000, the per cent. as computed; for numbers between 2000 and 3000, twice the per cent.; and for numbers above 3000, three times the per cent. The above was based on organisms of all kinds disregarding genera. An index of 50 would mean that organisms were noticeable half the time, or that if they were present for less than half the time they were more troublesome during the time when they were present." The maximum index of frequency possible by this method of computation is 300. Natural waters rarely have an index of more than 100. Best stored waters have an index less than 10. See Table 230.

Bacteria in Surface Waters. Most rivers in inhabited regions contain several hundreds or thousands of bacteria per c.c., dependent upon the degree of pollution. Furthermore, these numbers fluctuate rapidly, especially during storms and floods. Table 232 illustrates these seasonable variations.

Table 232. Seasonal Variations in Bacterial Content of River Waters. Bacteria per c.c., Monthly Average
(PRESCOTT AND WINSLOW)

River	Year	Jan	Feb	Mar.	April	May	June
Thames*	1905–6	2,075	1,679	1,161	277	1,064	382
Lea*	1905–6	5,192	3,083	1,308	471	1,350	598
New*	1905–6	1,455	1,304	291	149	352	198
Mississippi†	1900–01	972	2,871	1,795	3,597	2,152	2,007
Potomac‡	1906–7	4,400	1,000	11,500	3,700	750	2,300
Merrimac‖	1905	14,200	14,800	10,300	3,600	1,900	9,600
Susquehanna§	1906	9,510	21,228	31,326	39,905	6,187	2,903

River	Year	July	Aug.	Sept	Oct	Nov.	Dec.
Thames*	1905–6	952	————	————	————	1,633	740
Lea*	1905–6	1,190	————	————	————	3,946	2,050
New*	1905–6	450	————	————	————	718	621
Mississippi†	1900–01	1,832	805	————	————	————	2,021
Potomac‡	1906–7	2,700	3,000	6,200	2,300	1,800	6,900
Merrimac‖	1905	3,900	19,500	13,500	39,800	8,700	————
Susquehanna§	1906	685	1,637	836	7,575	26,224	37,525

* Houston, 1906*a*, 1906*b*.
† New Orleans, 1903.
‡ Figures obtained through courtesy of F. F. Longley.
‖ Massachusetts, 1906.
§ Harrisburg, 1907.

In warm weather and during periods of low flow, putrefaction is likely to occur in river water, greatly changing its bacterial content. This condition is accompanied by loss of oxygen, sometimes to its entire disappearance and the destruction of fish life, and may indirectly cause an increase in bacteria. Temperature itself has but slight influence upon the numbers of bacteria, although growths of antagonistic microscopic organisms are most frequent during the warmer months.

Discharge of sewage affects the bacteria markedly. Bacteria in the Ni-

agara river vary between 10,000 and 300,000 per c.c., and average 25,000 per c.c., dependent upon the degree of sewage contamination (E. R., **65**, 601, 1912). According to Miquel (Revue d'Hygiene, **8**, 1896, 388) there are in the Seine river above Paris 300 bacteria per c.c. and below Paris (Clichy) 200,000 per c.c. Koch* found in the Spree 82,000 bacteria per c.c. above Koepenick and 10,000,000 per c.c. at Charlottenburg some miles below. Schlatte (Zeit f. Hygiene, **9**, 1890, 56) found in the Limmat 1000 to 2000 bacteria per c.c. before the addition of the sewage of Wipkingen, and 5000 thereafter. The Spree contains 2,500,000 more bacteria per c.c. below Berlin than above. As in ground waters, the number, not the mere presence, of B. coli is an index of the degree of pollution. Almost every river water will give some positive tests for B. coli with 100 c.c. test volumes. Prescott and Winslow (*loc. cit.*) quote from Houston's study of the Thames at different points to illustrate increase and decrease in B. coli as the stream passes

Table 233. B. Coli in the River Thames at Various Points
(Houston, 1904)
Percentage of Positive Results

Place	+10 c.c.	+10 to 1 c.c.	+1 to 0.1 c.c.	+0.1 to 0.01 c.c.	+0.01 to 0.001 c.c.	+0.001 to 0.0001 c.c.	+0.0001 to 0.00001 c.c.
Sunbury				70.6	23.5	5.9	
Hampton			11.8	64.7	17.7	5.9	
Barking				4.2	45.8	45.8	4.2
Crossness				11.1	27.7	50.0	11.1
Purfleet			3.0	9.1	33.3	39.1	15.1
Grays			2.8	22.2	41.7	33.3	
Mucking			30.8	57.7	11.5		
Chapman	5.0	45.0	50.0				
Barrow Deep	12.0	36.0	40.0	12.0			

through London. Table 234 by J. W. Ellms (Report Cincinnati Water Purification Plant, 1914) shows fluctuations in bacteria and B. coli in Ohio river, also the relations between them. Table 234 gives results for the Ohio river, a muddy stream containing on the average 135 p.p.m. of suspended matter; Table 235 gives similar results for the clearer but more highly colored Delaware river, containing 36 p.p.m. of suspended matter. (West, Report of Phila. Bureau of Water, 1913.) (Table 239.)

Lake waters contain fewer bacteria than river waters, although polluted with sewage at the shores and at the mouths of rivers. In 1897 Weston found in Lake Superior near Duluth no bacteria at a depth of 100 ft., although the surface water, polluted by sewage from Duluth and West Superior, Wis., contained several hundred per c.c. Dr. E. Channing Stowell found the water of Dublin pond (N. H.) practically sterile at a depth of 40 ft., although B. coli were occasionally found along the shores. Table 236 is an illustration of the variations in bacteria and B. coli in Crystal Lake, Wakefield, Mass., for the year ending July 1, 1915.* This is an example of a water which is safe to

* Das Wasser, p 119 (Blücher, Leipzig, 1900).

Table 234. Bacteria in Ohio River Water for the Year 1914

(Ellms)

Month	No. of bacteria on gelatin at 20° C.*									Bacillus coli										B. coli index‡
	No. of test days	Mean per c c.	Median per c c	Variations in numbers: No. of test days						No. of test days	0.01 c c. tests			0 1 c.c. tests			1.0 c.c. tests			No. per c.c.
				0–100	101–300	301–1000	1001–10,000	10,001–100,000	Above 100,000		Total No.	No. +	Per cent. +	Total No.	No. +	Per cent. +	Total No.	No. +	Per cent. +	
Jan . .	31	51,710	38,000	0	0	0	0	27	4	31	31	3	9.68	31	19	61 29	31	31	100.00	16 21
Feb	28	53,820	50,000	0	0	0	0	26	2	28	28	3	10 71	28	15	53 57	28	28	100.00	15.46
Mar .	31	37,000	33,000	0	0	0	0	31	0	31	31	1	3 23	31	15	48.39	31	31	100 00	8.26
Apr.	30	11,420	8,150	0.	0	0	18	12	0	30	30	3	10.00	30	16	53 33	30	30	100.00	14.80
May	29	4,760	3,100	0	1	6	19	3	0.	31	31	5	16.13	31	15	48 39	31	29	93 55	19 81
June..	28	380	300	2	13	12	1	0	0	30	30	1	3 33	30	4	13 33	30	24	80.00	5.00
July. .	31	420	400	2	12	17	0	0	0	31	31	1	3.23	31	11	35.48	31	20	65.42	6.75
Aug ..	31	2,750	550	0	2	20	8	1	0	31	31	9	29 03	31	16	51 61	31	29	93 55	31.71
Sept .	28	1,240	365	0	12	7	9	0	0	30	30	3	10 00	30	12	40 00	30	26	86 67	13.47
Oct.	27	2,480	340	3	9	8	5	2	0	31	31	6	19 35	31	18	58.06	31	29	93.55	23.58
Nov . .	28	230	215	0	24	4	0	0	0	30	30	0	0 00	30	1	3 33	30	20	66 67	0 97
Dec.. .	30	31,720	27,000	0	0	0	4	26	0	31	31	4	12 90	31	28	90 32	31	31	100.00	20.74
Total	352*			7	73	74	64	128	6	365	365	39	.	365	170	.	365	328	. .	... ...
Average		16,500	†2,900								.		10 63			46.43	.	..	89.88	14 73
% Time.	96 44			1 99	20 74	21 02	18 18	36 36	1 71	100	100	10 69	... .	100	46 58	.	100	89.87	... .	... .

* No results on 13 days due to liquefaction
† True median, not an average
Forms recommended by Com. of N. E W W Assn
‡ Note.—B. coli index is reciprocal of largest test volume in which B coli were found in a sample It is an approximation of the number of B. coli present.

drink for a part of the time. It is only slightly polluted. In this lake, the organisms growing at 37° C. numbered as follows: Average bacteria per c.c., 72; maximum, 205; minimum, 56; median, 62.

Table 235. Bacteria in Delaware River Water at Torresdale, Pa., for the Year 1913 (West)

Month	No. of bacteria on gelatin at 20° C.						Bacillus coli						
	No. of test days.	Mean per c.c.	Median per c.c.	Variations in numbers No. of test days			No. of test days	0.1 c.c. tests			1.0 c.c. tests		
				300–1000	1000–10,000	10,000–100,000		Total No	No. +	Per cent. +	Total No.	No. +	Per cent. +
Jan.... ..	31	5,300	3,600	0	29	2	13	13	13	100	13	13	100
Feb	28	4,600	4,400	0	28	0	11	11	7	64	11	11	100
Mar	31	9,600	4,600	0	25	6	12	12	12	100	12	12	100
Apr. .	30	8,200	4,000	0	21	9	14	14	8	57	14	14	100
May .	31	6,700	5,200	0	27	4	12	12	11	92	12	12	100
June	30	7,300	2,900	1	24	5	13	13	10	77	13	13	100
July	31	5,000	3,400	3	27	1	14	14	10	71	14	13	93
Aug	31	9,000	6,200	1	22	8	12	12	11	92	12	12	100
Sept	30	5,600	4,200	0	28	2	13	13	10	77	13	13	100
Oct	31	10,000	7,000	0	24	7	13	13	13	100	13	13	100
Nov	30	8,500	6,200	0	24	6	11	11	11	100	11	11	100
Dec	31	12,000	7,700	0	22	9	15	15	15	100	15	15	100
Total	365			5	301	59	153	153	131		153	152	
Average. .		7,680	4,950										.
% Time ..				1	86	16			86			99.3	. ..

Table 236. Bacteria in Crystal Lake Water, Wakefield, Mass.
July 1, 1914–July 1, 1915

Month	No of bacteria on gelatin at 20° C.							Bacillus coli		
	No of test days	Mean per c.c	Median per c c	Variations in numbers: No of test days				10 c c tests		
				0–100	100–300	300–1000	1000–10,000	Total No	No +	Per cent +
Jan	15	709	680	0	3	10	2	15	7	47
Feb	10	220	212	0	2	8	0	10	2	20
Mar	4	181	178	0	4	0	0	4	0	0
Apr	4	149	146	0	4	0	0	4	0	0
May	3	172	170	0	3	0	0	3	1	33
June	2	285	285	0	1	1	0	2	1	50
July .	4	75	70	3	1	0	0	4	1	25
Aug	12	101	77	11	0	1	0	12	0	0
Sept	13	101	87	10	2	0	0 .	13	0	0
Oct	5	81	85	5	0	0	0	5	0	0
Nov	4	81	81	4	0	0	0	4	0	0
Dec	6	56	55	6	0	0	0	6	0	0
Total	81			39	20	20	2	81	12	
Average..		185	177				.			15
% Time				48	25	25	2		23	.

Lake Ontario near Toronto is an example of a polluted lake water. Results of examination of this water at Toronto for 1912 (Longley, Trans. Canadian Soc. C. E., Nov. 6, 1913) are given in Table 237.

Table 237. Bacteria in Lake Ontario at Toronto Waterworks, 1912
(LONGLEY)

Month	No. of bacteria on gelatin at 20° C. Max.	Min	Av.	Bacillus coli % positive, 10 c.c. tests
Jan.	7,200	119	2,163	0
Feb.	175,200	389	19,232	8
Mar	104,000	282	17,264	0
Apr	114,000	850	18,524	12
May	32,400	162	3,592	4
June	8,400	150	1,222	0
July		No results		
Aug.		No results		
Sept	1,140	50	325	8
Oct.	5,200	66	909	0
Nov	5,400	68	1,159	0
Dec.	32,000	40	5,988	0
Average.	48,494	218	7,040	
Average per cent., positive.				3.2

Suspended Mineral Matter. Most streams carry more or less suspended mineral matter, varying from several pounds to several tons per m.g. Quantities of suspended matter in various river waters are given in Table 243 (Samples 1 to 69), p. 668 and Table 25, page 49.

Table 238. Turbidity of Ohio River Water, 1914
(ELLMS)
(Parts per Million)

Month	No of test days	Mean turbidity	Median turbidity	Turbidity. Variations: No. of test days 0–10	11–25	26–50	51–100	101–250	251–500	above 500	Suspended matter
Jan	31	95	85	0	0	6	20	4	1	0	137
Feb	28	235	175	0	0	0	5	15	6	2	228
March	31	145	130	0	0	3	10	14	4	0	162
April	30	155	140	0	0	2	4	20	4	0	178
May	31	140	150	1	1	4	9	11	5	0	146
June	30	11	10	19	10	0	1	0	0	0	46
July	31	17	13	12	15	3	1	0	0	0	39
Aug	31	145	28	4	6	10	2	2	4	3	103
Sept	30	100	33	1	13	5	4	3	3	1	119
Oct	31	70	15	2	17	4	1	5	2	0	90
Nov	30	9	9	25	5	0	0	0	0	0	24
Dec	31	300	310	0	3	0	2	10	11	5	346
Total	365		Yearly median 60	64	70	37	59	84	40	11	
Average.		120	True median not an average*								135
% Time	100.00			17 53	19.18	10 13	16.16	23 02	10 96	3 02	

Coefficient of fineness = 1.12. Coefficient of fineness is the number obtained by dividing the weight of suspended matter in the sample (in parts per million) by the turbidity. If greater than unity, it indicates that the matter in suspension in the water is coarser than the standard; if less than unity, that it is finer. (Standard Methods of Water Analysis, A. P. H. A., 1915, p. 7.)
* See footnote on page 49.

Examples of variation in suspended matter, as shown by turbidity, are given in Table 238 for the Ohio and Table 239 for the Delaware at Torresdale.

Table 239. Turbidity and Color of Delaware River Water, 1913
(WEST)
(Parts per Million)

Month	No. of test days	Mean turbidity	Suspended matter	Turbidity. Variations: No. of test days					No of test days	Mean color	Color. Variations: No. of test days		
				0–10	11–25	26–50	51–100	101–250			0–10	11–25	21–50
Jan.	31	27	17	9	15	3	2	2	4	12	1	3	
Feb	28	11	18	17	10	1			4	16		4	
Mar. .	31	79	50		7	11	5	8	4	17		4	
Apr	30	31	49		18	8	3	1	5	15	1	4	
May .	31	19	33		27	3	1		4	15	1	3	
Jun .	30	14	22	8	22				4	15		4	
Jul. .	31	22	44	4	19	6	1	1	5	16		5	
Aug	31	21	29	7	14	10			4	18		4	
Sept	30	19	30	2	23	5			5	18		5	
Oct	31	38	50		6	19	5	1	4	19		4	
Nov	30	38	55		13	12	3	2	4	23		3	1
Dec	31	32	30		16	11	3	1	5	18		5	
Total	365			47	190	89	23	16	52		3	48	1
Average...		29 4	36										
% Time..–.				13	52	24	6	5			6	92	2

Coefficient of fineness = 1.22. Form recommended by Committee of N. E. W. W. Ass'n.

Odors in Surface Waters are due to: 1. *Organic Matter Other than Living Organisms.* Peaty, straw-like, swamp-like or marsh-like odor, usually grouped under the term "vegetable," is caused by vegetable matter, mostly in solution. Wastes from paper and textile mills, bleacheries, dye houses, gas works, chemical works, tanneries, etc., often impart a characteristic odor. 2. *Decomposition of Organic Matter.* Moldy, musty, unpleasant, disagreeable or offensive odors caused by decomposition are quite readily recognized. Moldy odor, suggesting a cellar in which vegetables are stored, is frequently observed in water from old and badly polluted, but not necessarily infected, wells. Musty odor occurs in a sewage polluted water. Vegetable matter, unless decomposed, rarely produces anything worse than an unpleasant odor. The odor called disagreeable is usually from animal sources, while the odor termed offensive may be produced by sewage or by decomposition of Cyanophyceæ. 3. *Living Organisms.* Natural odors caused by organisms are produced by volatile, oily compounds secreted by the organisms, the most notable one being set free at the time the organism disintegrates. When dead organisms decay and suffer bacterial decomposition, odors different from the natural odors are produced. These are all offensive. Cyanophyceæ produce a "pig pen" odor upon decomposition. See p. 666 under Ice. Whipple states that Synura "oil" may be detected in dilutions of 1:25,000,000.

Table 240. Number of Organisms per c.c. Required to Produce Noticeable Odor
(WHIPPLE)

Organism	Description of odor	No. per c.c.
Synedra....................	Vegetable and earthy	5,000
Cyclotella.................	Aromatic and fishy	5,000
Melosira.	Earthy and vegetable	3,000
Anabæna..	Rank vegetable	17
Scendesmus............. .	Vegetable and aromatic	25,000

The numbers of certain organisms per c.c. required to produce a noticeable odor are given in Table 240. Of 71 supplies taken from Massachusetts ponds and reservoirs, 45 were found to have given trouble, and 30 of these serious trouble, because of bad tastes or odors. Microscopic organisms are believed at present to be harmless. Many, however, are objectionable from an æsthetic standpoint and often indicative of abundant food material emanating from objectionable sources. Distinctive odors produced by certain organisms may be described by the three general terms "aromatic," "grassy" and "fishy," as tabulated in Table 241.

Table 241. Distinctive Odors Produced by Certain Organisms (Whipple)

Group	Organism	Natural odor
Aromatic odor.	Diatomaceæ:	
	Asterionella	Aromatic—geranium—fishy
	Cyclotella	Faintly aromatic
	Diatoma	Faintly aromatic
	Meridion	Aromatic
	Tabellaria	Aromatic
	Protozoa:	
	Cryptomonas	Candied violets
	Mallomonas	Aromatic—violets—fishy
Grassy odor . .	Cyanophyceæ:	
	Anabæna	Grassy and moldy—green-corn—nasturtiums, etc.
	Rivularia	Grassy and moldy
	Clathrocystis	Sweet, grassy
	Cœlosphærium	Sweet, grassy
	Aphanizomenon	Grassy
Fishy odor.....	Chlorophyceæ:	
	Volvox	Fishy
	Eudorina	Faintly fishy
	Pandorina	Faintly fishy
	Dictyosphærium	Faintly fishy
	Protozoa:	
	Uroglena	Fishy and oily
	Synura	Ripe cucumbers—bitter and spicy taste.
	Dinobryon	Fishy, like rockweed
	Bursaria	Irish moss—salt marsh—fishy
	Peridinium	Fishy, like clam-shells
	Glenodinium	Fishy

Ice is always purer than the water from which it is gathered. Winslow has shown that typhoid fever bacilli die rapidly in ice, and that ice stored for 5 months is safe, even though it be collected from polluted sources. Table 242 gives analyses of water and ice from a pond in Danvers, Mass. The water is not safe to drink but the ice may be used with impunity.

Growths of Occillaria, Anabæna and other organisms may become frozen in the ice of polluted ponds. Masses of these growths may decay and cause the formation of cavities within the ice containing dirty organic matter and gases which have a foul odor, often months after the ice is harvested.

Table 242. Analyses of Water and Ice from Same Source (Weston)

Source of sample	Water	Ice
Date of collection	April, 1915	April, 1915
Turbidity—silica standard	6.0	2 0
Color—platinum standard	21.0	0.0
Oxygen consumed	5.82	0 60
Nitrogen as free ammonia	0.026	0.124
Nitrogen as albuminoid ammonia (total)	0.200	0.080
Nitrogen as nitrites	0.001	0 001
Nitrogen as nitrates	0.70	0 00
Chlorine	18.60	0.64
Alkalinity	24.6	0.0
Hardness by soap method	40 8	1.6
Iron, Fe	0 15	0.05
Residue on evaporation (total)	101.0	20.0
Bacteria per c.c., 20° C.	390	7
Bacteria per c.c., 37.5° C	211	3
Bacillus coli in 10 c.c.	Positive	Negative
Bacillus coli in 100 c.c.	Positive	Negative

Results except bacteria expressed in parts per million.

Table 242a. International Atomic Weights (1912) of Elements Occurring in Water Analyses

	Symbol	Atomic weight		Symbol	Atomic weight
Aluminium	Al	27 1	Manganese	Mn	54 93
Arsenic	As	74 96	Mercury..	Hg	200.6
Barium	Ba	137 37	Molybdenum	Mo	96 0
Boron	B	11 0	Nickel....	Ni	58 68
Bromine	Br	79 92	Nitrogen	N	14 01
Calcium	Ca	40 07	Oxygen	O	16 00
Carbon	C	12 00	Phosphorus	P	31 04
Chlorine	Cl	35 46	Platinum	Pt	195 2
Chromium	Cr	52 0	Potassium	K	39 10
Cobalt..	Co	58 97	Radium	Ra	226 4
Copper	Cu	63 57	Silicon	Si	28 3
Fluorine	F	19 0	Silver	Ag	107 88
Hydrogen.	H	1 008	Sodium	Na	23 00
Iodine...	I	126 92	Strontium	Sr	87 63
Iron.	Fe	55 84	Sulphur	S	32 07
Lead	Pb	207 10	Tin	Sn	119 0
Lithium	Li	6 94	Zinc	Zn	65 37
Magnesium	Mg	24 32			

Table 243. Water Analyses;* Chemical and Physical
(Parts in 1,000,000)

RIVER WATERS—AMERICAN (SANITARY)

No	River	Place	Reported by	Turbidity	Color	Oxygen consumed	Nitrogen as: Free ammonia	Nitrogen as: Albuminoid ammonia, Total	Nitrogen as: Albuminoid ammonia, Suspended	Nitrogen as: Nitrites	Nitrogen as: Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation: Total	Residue on evaporation: Suspended	Residue on evaporation: Dissolved	Iron	Carbonic acid
1	Allegheny	Pittsburgh, Pa.	Drake......	85 0	6	3 20	0.060	0 150	— — —	0 006	0.22	17 0	48.0	19.0	220.0	— —	— —	2.0	— —
2	Connecticut	Middletown, Conn	Conn. State Bd Health	———	30	———	0.068	0 164	.....	0.008	0.18	2 7	32.0	.....	74 0	....	....	....	— —
3	Delaware..	Philadelphia, Pa...	West	———	— —	3.70	..	....	———	.	.	6 0	47 6	27.2	122 0	36	....	2.28	....
4	Hudson	Albany, N. Y...	Hazen	39 0	30	5.51	0 096	0.266	———	0 006	0.372	4 0	67.0	.....	144 0	— —	— —	— —	....
5	James	Richmond, Va	Mason	...	.	1.65	0 550	0 150	———	tr.	tr.	1 17	———	.....	105 0	— —	— —	....	....
6	Kennebec	Augusta, Me.......	G C. Whipple	5.3	32	.....	0.019	0.125	———	0.001	0.04	1.06	19.1	15.3	57.8	....	....	— —	— —
7	Merrimac	Lawrence, Mass....	Mass State Bd. Health	.....	30	.....	0 089	0 234	.....	0.002	0 060	2.5	11.0	.....	39.6	— —	— —	— —	....
8	Mississippi	Minneapolis, Minn.	A D. Meeds..	———	40	7 60	0 072	0 240	———	tr.	tr.	2 0	164.0	150 0	197.0	— —	— —	— —	....
9	Mississippi	New Orleans, La.	R. S. Weston	423.0	13	6 9	0.006	0 245	———	0 00	0.12	9 3	84.0	75 0	573.0	— —	— —	— —	— —
10	Missouri	Omaha, Neb	Teichman	———	— —	7 1	0.062	———	———	0 004	0 18	10.0	———	———	1325.0	— —	— —	— —	....
	Ohio	Cincinnati, Ohio...	G. W. Fuller: ...																
11	Maximum			———	— —	46 0	0.074	0 868	0.758	0.030	1.34	———	57.0	70 0	2556 0	2333	223	— —	47
12	Minimum			———	— —	13.0	0 008	0 106	0 022	0 000	0 37	———	11 0	20 0	91 0	24	67	— —	6
13	Normal			———	— —	24 0	0 025	0 290	0 200	0 003	0 60	———	33 0	45 0	350 0	230	120	— —	26
14	Potomac	Washington, D C	R. S. Weston	80 0	5	2 60	0 008	0 111	...	0 002	0.11	4 0	———	— —	140 0	45	95	— —	— —
15	Schuylkill	Philadelphia, Pa	Mason	———	— —	———	0 010	0 100	.	0 000	0 46	———	———	———	133.4	— —	— —	— —	— —

* See Standard Methods, Am. P. H. Assoc , abstracted on p. 788.

RIVER WATERS—FOREIGN (SANITARY)

No.	River	Place	Reported by	Oxygen consumed	Nitrogen as: Free ammonia	Nitrogen as: Albuminoid ammonia, total	Nitrogen as: Nitrites	Nitrogen as: Nitrates	Chlorine	Hardness by soap	Residue on evaporation, total
50	Oder	Breslau	Poleck	15 4	0 06	——————	————	1.2	7.0	———	135
51	Rhine..	——————————	Vohe.	———	—————	——————	————	tr.	2.5	———	250
52	Spree	Berlin...	Tiemann	7 52 / 10.88	0 2	0 3	0.00	0.11	21.3	———	172 / 233
53	Thames	Sudbury, England..	Frankland..	2.01	0 006	0 330	————	2.10†	7.73	204	280

† This figure represents nitrites + nitrates

Table 243. Water Analyses; Chemical and Physical.—*Continued*

River Waters—American (Mineral)

No.	River	Place	Turbidity*	Coefficient of fineness*	SiO_2	Fe	Ca	Mg	Na+K	NO_3	CO_3	HCO_3	Cl	SO_4	Residue on evaporation: Suspended	Residue on evaporation: Dissolved
16	Alabama	Selma, Ala	141	0 72	21 0	0 53	13 0	2 9	7 0	0 7	0 0	48	2 3	9 0	100 0	82
17	Allegheny	Kittanning, Pa	21	1 14	7 9	0 13	14 0	3 0	11 0	0 7	0 0	38	14.0	17 0	30.0	87
18	Androscoggin	Brunswick, Me.	-----	-----	9 0	3 2	7 4	1.1	35 0	..	...	.....	2.3	12.0	-----	42
19	Arkansas	Little Rock, Ark	755	0 88	28 0	0 82	55 0	13 0	144 0	2 0	0 0	148	203.0	93.0	748.0	630
20	Chattahoochee	West Point, Ga	185	0 71	20 0	0 47	4 8	0 8	7 7	0 7	0 0	23	2.1	4 5	136 0	52
21	Colorado	Austin, Texas	.....	.....	18 0	3.10	52 0	17 0	49 0	.	0 0	195	59.0	42 0	351 0	321
22	Delaware	Lambertville, N. J.	16	1 74	9 0	0 07	12 0	3.3	5 4	1.1	0 0	46	2 9	12 0	26 0	70
23	Hudson	Hudson, N. Y.	13	1.26	11 0	0 15	21.0	3.8	7.9	0 8	0 0	73	4 0	16 0	16 0	108
24	James	Richmond, Va	90	0 96	18.0	0.50	14 0	3 0	6.7	0 3	0.0	60	2 3	7 1	71.0	89
25	Mississippi	Minneapolis, Minn	10	1 13	15.0	0 07	40 0	14 0	10 0	1.4	0 0	188	1.6	18 0	7 9	200
26	Mississippi	Memphis, Tenn.	556	0 97	24 0	0 61	36 0	12.0	19 0	1 7	0 0	129	8 6	43.0	634 0	115
27	Mississippi	New Orleans, La	-----	-----	11 0	0 53	32.0	8.4	13 0	2.5	0.0	111	9 7	24 0	-----	71
28	Missouri	Kansas City, Kan.	1909	1.04	37.0	0 73	62 0	18 0	44 0	2 2	0 0	202	13.0	135 0	2032 0	426
29	Potomac	Cumberland, Md	28	1.59	8 2	0 14	24 0	4.6	9 0	0 9	0.0	36	6 4	58 0	29.0	130
30	Rio Grande	Loredo, Texas	2475	.....	29 0	3 6	104 0	23 0	119 0	.....	0 0	178	164.0	228.0	.....	791
31	Sacramento	Sacramento, Calif	66	1 55	28 0	0 43	13 0	6 7	13 0	0 64	0 0	74	5.4	17 0	102 0	113
32	St Lawrence	Ogdensburg, N Y.	45	.....	6 6	0 05	31 0	7 2	6 3	0 3	2 9	116	7.7	12.0	.....	134
33	Tennessee	Knoxville, Tenn.	204	0 81	25 0	0 54	22 0	4 3	8 2	0 8	0 0	86	9 6	6 4	156.0	122

No. 16–33 Reported by U S Geol. Survey, Water Supply Papers, Nos 79, 236 and 237. * See also Table 25, page 49.

River Waters—Foreign (Mineral)

No.	River	Place	SiO_2	Ca	Mg	Na	K	NO_3	CO_2	Cl	SO_4	Total residue
34	Danube	Vienna	5	34	7				61	5	13	141
35	Elbe	Hamburg	5	28	1				45	5	7	127
36	Garonne	Toulons	40	26	1	6	3		45	2	8	136
37	Loire	Orleans	41	19	2	9	6		42	3	2	134
38	Rhine	Basle	2	55	5	1			86	2	15	169
39	Rhone	Lyons	7	66	1				90		20	184
40	Spree	Berlin		26	3	2	2	3	45		9	114
41	Thames	London Bridge	2	82		14	2		69	64	32	408

No.	River	Place	CaO	MgO	SO_4	$Al_2O_3 + Fe_2O_3$	Cl	$KNO_3 + NaNO_3$	Total residue	Suspended residue	Volatile residue
42	Elbe	Surface	40	8 0	7	4	24	......	138	72	46
43	Elbe	Bottom	46	14 0	8	6	16		151	58	49
44	Oder	Breslau	29	8 1	14	..	7	1.2	135	......	......

No. 34–41, Stillman: Engr. Chemistry (Chemical Pub, Co 1900). No 42 and 43. Reported by J Jettman. No. 44. Reported by Poleck.

* See page 664.

Table 243. Water Analyses; Chemical and Physical.—(*Continued*)

RIVER WATERS—FOREIGN (MINERAL).—(*Continued*)

No.	River	Place	SiO_2	$CaCO_3$	$MgCO_3$	$CaSO_4$	$MgSO_4$	$K_2SO_4 + Na_2SO_4$	$Fe_2O_3 + Al_2O_3$	NaCl	Cl	$KNO_3 + NaNO_3$	$CaCl_2$	Total residue
45	Danube	Vienna	4.9	83.7	15.0	2.9	15.7	2.0	2.0	1.0	5.8	......	......	126.2
46	Rhine	Strassburg	48.8	135.6	5.1	14.7	......	13.5	14.1	2.0	......	3.8	......	231.8
47	Rhone	Geneva	23.8	78.9	4.9	46.6	6.3	7.4	3.7	1.7	......	8.5	......	182.0
48	Seine	Above Paris	......	92.0	39.0	20.0	*	*	8.0†	......	......	......	10.0	179.0
49	Spree	Berlin	......	65.0	9.0	0.0	0.0	18.0	......	12.0	......	3.0	......	114.0

* $MgSO_4 + K_2SO_4 + Na_2SO_4 = 10.0$. † Includes SiO_2. No. 45–49. From Philips Eng. Chemistry (Crosby, Lockwood and Son, 1902).

Table 243. Rain and Snow; Springs and Wells; Ice

No.	Kind	Place	Reported by	Character	Oxygen consumed	Nitrogen as: Free ammonia	Albuminoid ammonia, total	Nitrites	Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	CO_2*	SiO_2
69	Rain	Lawrence, Mass.	Mass. State Bd. Health Report 1890	— — —	......	0.298	0.0024	0.0000	0.000	0.007	......	......	......	....	....
70	Rain	Jamaica Plain, Mass.	"	— — —	......	0.056	0.0152	0.0004	0.018	0.130	......	......	......	....	....
71	Rain	London, Eng.	Mason	— — —	0.99	0.50	0.22	......	0.07*	6.30	......	......	39.5	....	....
72	Snow	Boston, Mass	Mass. State Bd. Health Report 1890	— — —	......	0.025	0.0038	......	0.003	......	......	......	......	....	....
73	Snow	Troy, N. Y	Mason	— — —	......	0 046	0.022	0.00	0.00	0.188	......	......	......	....	....
74	Snow	London, Eng.	Mason	— — —	......	0.06	0.021	0.024	0.02	1.08	......	......	16.0	....	....
75	Dirty cistern water	West Troy, N. Y.	Mason	Bad	2.25	1.050	0.175	Strong trace	0.00	2.00	......	......	20.0	....	....
76	Shallow well	Elgin, Ill.	Ill. State Water Survey	Good	0.6	0.076	0 058	0.000	2 00	6.	......	210.0	298.0	....	....
77	Shallow well	Elgin, Ill.	Ill. State Water Survey	Bad	2.0	0.032	0.152	0.002	8.00	200.	......	388.0	973.0	....	....
78	Deep well	Champaign, Ill.	Ill. State Water Survey	Good	4 3	1.600	0.160	0.000	0.08	3.	......	378.0	398.0	....	....
79	Deep well	Moosup, Conn.	Conn State Bd. Health	Bad	0.24	0.028	0.038	0.019	11.2	17.7	142.0	44.0	229.0	....	....
80	Artesian well	Pomfret, Conn.	R. S. Weston	Good	0 040	0 010	0.026	0.003	0 020	2.62	65.7	64.5	123.0	....	....
81	Artesian well	Gloucester, Mass.	R. S. Weston	Bad	8.67	36 06	12.84	0 040	10 4	1200.0	335.0	1.9	2770.0	....	....
82	Dune	Amsterdam, Hol.	Mason-Eng News	Good	....	0 19	0.106	0.0	0.33	32.3	......	......	344.0	96.8	7.6
83	Spring	Portland, Conn.	Conn. State Bd. Health	Good	0.1	0 004	0 008	0 0	0 1	3.04	32.0	20.0	56.0	....	....
84	Spring	Brooklyn, Conn.	Conn. State Bd. Health	Bad	0.34	0 024	0.044	0.011	4.8	20.8	99.0	......	202.0	....	....
85	Ice pond	Wellesley, Mass.	R. S. Weston	Bad	....	0.024	0.218	0.008	0.2	9.5	35.1	29.0	96.0	....	....
86	Ice	Wellesley, Mass	R. S. Weston	Good		0 078	0.104	0.007	0 00	0.5	9 5	1.0	12.0	....	....
87	Ice pond	Danvers, Mass.	R. S. Weston	Bad	5 82	0.026	0 200	0.001	0.70	18.60	40.8	24.6	101.0	....	....
88	Ice	Danvers, Mass.	R. S. Weston	Good	0 60	0.124	0.080	0.001	0 00	0.64	1.6	0.9	20.0	....	....

* This figure represents nitrates and nitrites.

Table 243. Water Analyses; Chemical and Physical.—*Continued*

LAKE AND RESERVOIR WATER

a-AMERICAN (SANITARY)

No.	Lake	Reported by	Turbidity	Color	Oxygen consumed	Nitrogen as Free ammonia	Nitrogen as Albuminoid ammonia, total	Nitrogen as Nitrites	Nitrogen as Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	Iron	Sulphate	CaO	MgO
54	Champlain	U. S. Geol. Survey	---	—	---	0.031	0.132	0.002	0.092	0.9	50.6	---	65.6	....	---	--	---
55	Cochituate	Mass. State Bd. Health	.....	24	4.2	0.029	0.221	0.001	0.027	5.4	20.0	---	52.1	....	---	--	---
56	Croton	Report of Comm. on additional water supply for N. Y.	6	27	---	0.077	0.191	0.002	0.10	1.7	39.4	31.0	71.0	0.23	---	--	---
57	Erie	Mason	.....	..	1.25	0.045	0.112	tr.	0.08	3.5	......	......	134.0	....	---	--	---
58	George	N. Y. State Bd. Health	clear		2.0	0.066	0.062	0.001	0.04	1.0	12.7	6.0	38.0	....	---	--	---
59	Michigan*	Gehrman	---	00	7.1	0.024	0.044	0.00	0.200	5.0	93.0		140.0	0.4	---	--	---
60	Superior†	Mason	.....	..	1.15	0.03	0.02	0.00	0.00	2.0	......	......	54.0	....	---	--	---
61	Wachusett Reservoir	Mass. State Bd. Health	.....	18	3.1	0.022	0.161	0.001	0.024	0.25	8.0	.....	30.3	....	---	--	---
62	Winnepesaukee	U. S. Geol. Survey	.....	10	.....	0.002	0.093	0.000	0.038	1.2	....	.	20.9	...	---	--	---

b-FOREIGN (SANITARY)

No.	Lake	Place	Reported by	Turbidity	Color	Oxygen consumed	Nitrogen as Free ammonia	Nitrogen as Albuminoid ammonia, total	Nitrogen as Nitrites	Nitrogen as Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	Iron	Sulphate	CaO	MgO
63	Geneva	....	"Das Wasser"	.....	..	.....	.....	.....	.....	.....	.....	.....	.....	128.0	....	38.1	42.3	11.5
64	Katrine	Scotland	Frankland	.....	..	1.86	0.00	0.110	.....	0.04§	6.7	9.5	.....	29.6	....	.....	....	.....
65	Mügel	Berlin	Wehner	.....	.	3.87	0.28‡	.....	0.00	0.00	5.6	126.0	.....	243.0	...	16.8	....	.....
66	Tegel	Berlin	"Das Wasser"	.....	.	12.9	0.70‡	.....	.....	2.1	12.6	.....	. .	188.3	....	10.2	63.5	81.0
67	Thirlmere	Manchester, Eng. Supply	Frankland			1.74	0.080	0.170	0.00	0.00	7.5	9.0	.	23.6	..	.....	.....	.....
68	Zürich	....	"Das Wasser"		.	....	.....	.....	.....	.....	.....	.....	.	140.6	..	9.2	56.6	10.0

* Four miles out at Chicago Intake.
† Near Duluth.
‡ These figures represent free ammonia + albuminoid ammonia.
§ This figure represents nitrites + nitrates.

Water Analyses; Sea Water

No.	Sea	Reported by	Ca	Mg	Na	K	Nitrogen as NH₄	Al	Fe	Mn	CO₃	SO₄	Cl	Br	Residue on evaporation, total
89	Atlantic Ocean	Stillman Eng. Chemistry	557	1,198	17,720	668	.....	....	.....	....	.....	3,030	20,840	388	38,400
90	Pacific Ocean	Stillman Eng. Chemistry	475	1,471	10,233	634	.....	....	.....	....	.....	2,827	19,321	239	35,200
91	Baltic Sea	Stillman Eng. Chemistry	36	612	5,894	......	.....	....	.....	....	.....	719	10,386	......	17,710
92	Black Sea	Stillman Eng. Chemistry	131	662	5,512	98	.....	....	127.0	....	248	1,250	9,575	5	17,608
93	Caspian Sea	Stillman Eng. Chemistry	192	410	1,444	140	.....	40	.....	....	77	1,337	2,738	......	6,296
94	Dead Sea	Stillman Eng. Chemistry	9,000	19,883	47,918	6,385	18	153	12.0	26	.....	471	154,442	2,177	240,483
95	Mediterranean Sea	Stillman Eng. Chemistry	444	1,310	11,706	264	.....	....	2.8	....	68	2,943	20,527	434	37,706

CHAPTER XXXI

INSPECTION OF SOURCES OF WATER SUPPLY

Origin and Longevity of Polluting Bacteria. To safeguard the purity of water supplies, careful and proper inspection should be made in order to eliminate any possible source of contamination. Chief sources of contamination are sewage of cities and towns and willful or careless discharge of dejecta by individuals. Water-borne diseases comprise those caused by bacteria carried from the intestine of one individual to the mouth of another. Intestinal bacteria, including B. coli and the typhoid bacilli, tend to die out in natural waters. According to various authorities (Gärtner: Klin. Jahrb. (1902), **9**, p. 335; Jordan, Russell, and Zeit: Jour. Infec. Diseases (1904), **1**, p. 641; and Houston: "Studies in Water Supply," p. 83 *et seq.*) over 95 per cent. of typhoid bacilli discharged into surface water die during the week following their entrance, and after 1 month they have practically ceased to exist. (See Tables 247 and 248, page 681.) The surviving minority may persist as long as 2 months. Experiments have shown that typhoid bacilli perish more rapidly in polluted surface waters than in unpolluted ground waters. Jordan, Russell and Zeit (*loc. cit.*) attribute this to the antagonism of normal water bacteria. Protozoa also consume bacteria. Houston has shown that uncultivated typhoid bacilli as they are discharged by typhoid patients die more rapidly in Thames river water than those which had been isolated and grown in the laboratory on artificial media.

Importance of Time. From a sanitary standpoint the discharge of many million bacteria by one typhoid patient into a water about to be distributed is far more serious than the discharge of the sewage of a city, if the contaminated water be stored for a month or two before use. This consideration should enter into all rules and regulations for the protection of a source of supply.

River Supplies. Purification of river water by filtration should be supplemented by careful inspection of the river, especially above the waterworks intake, where it should be protected from all pollution or contamination. Storage reservoirs are always advantageous, and should be carefully inspected to prevent their pollution. Filter beds and filter sand should be guarded carefully to prevent accidental pollution, and all filtered water basins, suction wells and pipes should be tight to prevent contamination by infected ground water (Jordan and Irons, Jour. of Infec. Dis., **11**, July, 1912, on Rockford Epidemic). Distributing reservoirs containing water about to be used should be carefully guarded.

Lake and Reservoir Supplies. Long storage coupled with careful inspection can produce a fairly safe water even from polluted sources, as the Lake Cochituate, Croton* and Thames river drainage areas. While modern com-

* Croton water has, however, for a number of years been treated with hypochlorite of lime or liquid chlorine put into the aqueducts at a place relatively near New York City. Many engineers who have studied this water believe it should be filtered for both sanitary and physical improvement.

munities are demanding water which is safer and better in appearance than can be obtained without filtration, inspection can accomplish much in reducing dangers of infection.

Lake sources should be frequently inspected and inspection supplemented by numerous and regular bacteriological analyses.

Water traffic on the Great Lakes is a grave source of danger, and lake craft should be required to disinfect discharges from toilet rooms. This can be accomplished by heating with steam or by addition of disinfectants to the sewage before discharge (See Metcalf and Eddy, "American Sewerage Practice," 1915, vol. III p. 737).

Rules and regulations adopted by Metropolitan Water and Sewerage Board of Massachusetts are in outline as follows:

No polluting matter of any kind shall be discharged directly into a water supply. No cesspool, privy or other receptacle for house drainage or polluted water shall be located within 250 ft. of the high-water mark of any reservoir unless such cesspool, privy or other receptacle be so constructed that no portion of its contents can escape into the water. No garbage, manure or putrescible matter of any kind shall be put into the water supply or upon the ground within 250 ft of high-water mark, except that used in the cultivation and use of soil in the ordinary methods of agriculture.

No stable, pig sty, henhouse, barnyard, hog yard, paddock for horses, cattle or other animals, shall be located or maintained within 250 ft of high-water mark unless suitable provision is made to prevent any manure or other polluted matter from being washed into the water

All hospitals, tanneries, currying shops, and slaughter houses must be located as approved by the State Dept of Health, and cannot be maintained on the drainage area unless they comply with all the provisions required by said Dept for the purification or disposal of sewage, drainage or other polluting or organic matter which may be discharged therefrom.

No persons shall bathe in the reservoirs.

Rules and regulations of the Board of Water Supply of the City of New York, duly approved by the State Department of Health, promulgated under Chapter 665 of the Laws of 1915 for protection against pollution of the waters of Ashokan reservoir and tributary watercourses:

Definitions. "The City" shall mean The City of New York or its duly authorized representative "Watercourse" shall mean any natural or artificial spring, stream or channel of any kind, in which water flows continually or intermittently over any part of the watershed to one of The City's reservoirs. "Reservoir" shall mean any natural or artificial lake, pond or reservoir within the limits of a watershed used by The City for its water-supply.

Rules and Regulations. Sewage containing human excreta (1) No privy or receptacle of any kind for the deposit, storage, removal or disposal of human excreta shall be constructed, located or maintained in such manner as to pollute or threaten the pollution of any watercourse or reservoir

(2) All transportable receptacles for human excreta shall be provided with tightly fitting covers, which shall be securely fastened during removal and transportation, so that no portion of the contents can escape. A sufficient number of receptacles shall be provided, so that whenever one is removed an empty receptacle can at once be substituted. The receptacles shall be thoroughly cleansed and disinfected, as often as may be found necessary in order to maintain proper sanitary conditions.

(3) In the absence of some manner of disposal of excreta specifically approved by the Board of Water Supply after due submission of plans thereof to said Board, such excreta shall be disposed of by burial in shallow trenches not more than 2 ft. deep, in places well removed from all watercourses, where the flatness of the ground, the character of the subsoil and the depth of the ground-water level will afford ample security against contamination of any watercourse or undue pollution of the ground-water and the soil.

(4) In approved locations and where conditions of soil and topography are favorable, water-flushed toilets and other inside plumbing may be used, if connected by approved water-tight pipes with an approved sewage-disposal system.

House Slops. (5) House slops, sink wastes, laundry water or sewage of any kind shall not be thrown on the ground or discharged in such manner as to become offensive or to result in or threaten the pollution of a watercourse or reservoir Whenever large quantities of polluting solid matter or polluted liquid are to be disposed of, application shall be made to the Board of Water Supply for the approval of a satisfactory method of disposal.

Garbage, Refuse, Compost, Manure and Dead Animals. (6) No garbage or putrescible refuse of any kind shall be thrown or discharged directly into any watercourse or reservoir, nor shall the disposal be such as to result in or threaten pollution

(7) Compost heaps, or masses of fermented or decayed foods, vegetables, roots, grain, sawdust, leaves or other vegetable substances shall not be maintained in such manner as to result in or threaten pollution.

(8) No dead animal of any kind, nor any part thereof, shall be thrown into any watercourse or reservoir or buried in such manner as to form a source of pollution.

Stables and Slaughter-houses. (9) No stable, pigsty, chicken house, barnyard, hog yard, slaughter-house, hitching or standing place of horses and cattle or any other place where dung or urine accumulates shall be constructed, located and maintained so as to form a source of pollution.

Factory Wastes. (10) No filth, decaying or putrescible matter, waste product or polluted liquid from any factory, creamery, mill, tannery, garage or establishment of any other kind shall be discharged, drained or washed directly into any watercourse or reservoir. Application shall be made to the Board of Water Supply for the approval of a satisfactory method for the disposal of such objectionable matters.

Washing, Bathing and Swimming. (11) No animals of any kind shall be washed in any watercourse or reservoir owned by The City. No clothes or other articles shall be washed in any watercourse or reservoir.

(12) No bathing, swimming nor washing shall be done in any watercourse or reservoir owned by The City.

Cemeteries. (13) No interment shall be made in any cemetery or other place of burial in such manner as to result in pollution of a watercourse or reservoir.

Disposal Systems. (14) Every existing system for treating excremental matter, wastes or discharges of any kind from any dwelling, hotel, stable, garage, factory or other building, wherein such wastes or discharges constitute a source of pollution, shall be modified in a manner satisfactory to the Board of Water Supply.

(15) All new systems for the treatment of excremental matter, wastes and discharges of any kind shall receive the approval of the Board of Water Supply before such system is installed.

NOTE.—These rules were prepared especially for the Catskill Mountain watersheds, where the slopes are very steep, the soil generally very shallow and impervious, and the habitable lands along the streams so narrow in many places that rules with set distances (like those of Met. W. & S. Bd. above) would be inapplicable.

Sanitation of contractor's camps and control of catchment areas are especially important, particularly when water, as is often the case, is used during construction of a reservoir, before the reservoir has filled sufficiently to provide adequate storage for purification. Supervision of camps has been most refined in connection with the Catskill water supply for New York City. Where camps were located within drainage areas utilized by towns, all camp drainage was filtered and disinfected. In most cases, all organic waste was consumed in incinerators, an inexpensive and reliable method of disposing of much offensive and dangerous matter.

The contracts contained the following paragraphs:

"The contractor shall take satisfactory precautions to prevent the contamination of, or interference with, any public or private water supply by his work or by his employees, and shall provide acceptable substitute supplies in case of unavoidable interference. The contractor shall furnish and erect, when, where and as directed, manproof fences along and adjacent to streams, reservoirs, etc., used as sources of potable water supply.

"All houses occupied by employees shall be thoroughly screened to exclude mosquitoes and flies. Quarters for the men shall be grouped in properly arranged camps. Camps shall, if ordered be enclosed by satisfactory manproof fences with not more than two entrances and the entire grounds illuminated by electric arc lamps or other acceptable lights. The contractor shall retain the services of acceptable, qualified, medical and surgical practitioners, to the number ordered, who shall have the care of his employees, shall inspect their dwellings, the stables and the sanitaries as often as required, and shall supply medical attendance and medicine to the employees whenever needed. They shall give their whole time to the work; there shall always be at least one on duty. . . . The contractor shall provide from approved plans one or more buildings properly fitted for the purposes of a hospital with facilities for heating and ventilating in cold weather. These hospitals shall have an ample number of beds to care properly for sick and injured employees, and shall be provided with all necessary medicines and medical appliances for the proper care of the sick and injured. Another building of approved design shall be provided and equipped as an isolation hospital, and any employee who shall be found to have a communicable disease shall be at once removed from the camp to this hospital and there isolated and treated as directed. Whenever practicable an employee having a communicable disease shall be removed when and as directed to an approved permanent hospital.

"Once a week, or more frequently, if required, the contractor shall give the engineer, in such detail as may be prescribed from time to time, a written report, signed by the physician in regular attendance, setting forth clearly the health condition of the camp or camps and of the employees If any case of communicable disease be discovered, or any case of doubtful diagnosis, it shall be reported at once to the engineer, by telephone or messenger and confirmed in writing.

"All wash water from kitchens, laundries and other places, and all drainage from stables, shall be conveyed by satisfactory means to places directed, where such drainage shall be treated by means ordered so as to yield an acceptably innocuous effluent.

"Drainage from camps and tunnels and from other places yielding water unfit for direct discharge into a reservoir or tributary thereof shall be conducted in tight drains or other approved conveyors to filters, septic tanks or other disposal plants of approved construction, at places designated, and treated as directed to produce an acceptable effluent. Such effluent shall be discharged only in the manner and at the place or places directed.

"Garbage, both liquid and solid, shall be promptly and satisfactorily removed from the buildings and immediately placed in approved tight receptacles of sufficient capacity for about one day's ordinary production. At least once in every twenty-four hours all such garbage shall be incinerated or otherwise thoroughly and satisfactorily disposed of in an approved manner.

"The contractor shall build, in accordance with drawings and directions furnished from time to time by the engineer, such disposal plants, sewers, drains and other structures, and shall do such other work, not herein particularly specified, as may be ordered for carrying out the intent of the sanitary precautions of the contract."

Whether *bathing, boating or fishing* should be allowed in a reservoir should be determined largely by the period of storage elapsing before use. In lakes or reservoirs remote from the distribution system, boating and fishing may be permissible, but should be restricted by permits. Bathing offers serious danger because of the chance of pollution by typhoid urine and feces, especially the former. So offensive is the thought of drinking water previously used for bathing that the practice should be prevented wherever practicable. Fishing and boating should be discouraged.

Ground water supplies should be inspected to see that there is no chance for contamination from surface sources (see Wells, page 654). Some ground waters of unimpeachable quality become polluted after being drawn to the surface, due to defective pump-wells or suctions. Ground waters which become infected are dangerous for a longer period than are surface waters and therefore should be inspected with equally great care. Inspection of sources and chemical and bacteriological examinations of samples should go hand in hand.

Disinfection of excreta and contaminated surroundings is always desirable.

Table 244. Disinfectants

Disinfectant	Solution for ordinary use, %	Proportion of chemical to disinfect excreta
Quicklime (CaO)	15.0	Equal volumes of excreta and 15 per cent. milk of lime.
Chloride of lime (bleaching powder) (calcium hypochlorite).	4.0	Equal volumes of excreta and 4 per cent. solution.
Phenol (carbolic acid)	5.0	Equal volumes of excreta and 5 per cent. solution, or 5 per cent. phenol in the final mass.
Cresol	2.0	Two per cent. cresol in the final mass.
Formaldehyde, "Formalin" = ± 30 per cent. solution of Formaldehyde.	5.0	Five per cent. formaldehyde in final mass.
Mercuric chloride	$\begin{cases} 0\ 25 \\ 0.1 \end{cases}$	Equal volumes of excreta and 1:500 solution.
Zinc chloride	10.0	Equal volumes of excreta and 10 per cent. solution.

Numerous coal-tar products, many of them very efficient, are used as disinfectants. Their strengths are given in terms of the "*Phenol Equivalent,*" obtained by dividing the figure indicating the degree of dilution of the disinfectant that kills specific bacteria in a given time by that expressing that degree of dilution of pure phenol which kills the same organisms in the same time and under parallel conditions. *Soap* increases the efficiency of coal-tar disinfectants, and cleanliness is safer than inadequate disinfection.

CHAPTER XXXII

STORAGE OF WATER AND IMPROVEMENT OF RESERVOIRS

Quality of water is greatly affected by storage, both for good and bad. Storage affords opportunity for coagulation, precipitation, bleaching of color, death of exotic disease germs and subsidence of silt and clay. Sometimes, however, stored waters develop growths of microscopic organisms, especially if the sources contain an abundant supply of food material or the water be stored in a new reservoir from which the vegetation and other organic matter have not been adequately removed. If the run-off be from a clean drainage area and the water be stored sufficiently long in an organically clean reservoir of adequate depth, a satisfactory supply usually results.

Period of storage in natural lakes may be many years, as in the Great Lakes, and may result in complete clarification and decolorization. Silver Lake, Brockton, Mass., has a storage capacity of 1766 days or 1,579,800 gals. per sq. mi. of drainage area; the color of the water is reduced from about 100 to 9 p.p.m. Periods of storage in certain other reservoirs are given in Table 246. At least 60 days' storage should be provided where surface waters are used without other means of purification. Even this nominal period may not afford sufficient protection against pollution because of imperfect displacement of the reservoir contents due to differences in the density of the water, due in turn to temperature changes and to the action of wind and currents.

Limnology, or the science of lakes and ponds, treats of their geology, geography, physics, chemistry, biology and the mutual relations of these features. (See Whipple, "Microscopy of Drinking Water," 1914, Chap. VII.)

Lake Thermometry. For surface temperatures, an ordinary chemical thermometer may be used, graduated from 0° to 40° C. or from 20° to 120° F. For subsurface temperatures at depths of less than 50 ft., this thermometer may be put in a weighted case enclosed in a stoppered bottle, lowered to the proper depth and filled by withdrawing the stopper. After allowing sufficient time for the thermometer to set, the bottle is drawn to the surface and the thermometer read before it is taken from the bottle. For deeper, as well as for shallower, temperatures the *thermophone* of Warren and Whipple (Whipple, "Microscopy of Drinking Water," 1914, p. 87), an electrical thermometer of the resistance type, is better adapted than any other instrument for taking series of observations. It (Fig. 347) consists of a resistance coil attached to the end of a 3-wire cable, which is lowered to the desired depth. Two wires are connected by leading wires to the terminals of a slide wire which forms part of the indicator. The third leading wire connects with a movable contact on a sliding wire having in its circuit a telephone and a current interrupter. These latter serve as a galvonometer to detect the presence of a current. The knob turning the pointer of the indicating dial is connected with the movable con-

tact. All parts except coil and cable are enclosed in a box. To take a reading, the coil is lowered to the desired depth, the current from the battery is turned on, the telephone is held to the ear and the indicator is moved to the point of silence in the telephone. At this point the hand indicates the temperature of the resistance coil. Ordinary thermophones, having a range of $-15°$ to $115°$ F., read correctly to 0.1° F. and may be used at depths of from 400 to 500 ft. Resistance coils may be permanently located in dams, etc., and connected with the indicating apparatus whenever a temperature reading is desired. (See also page 121.)

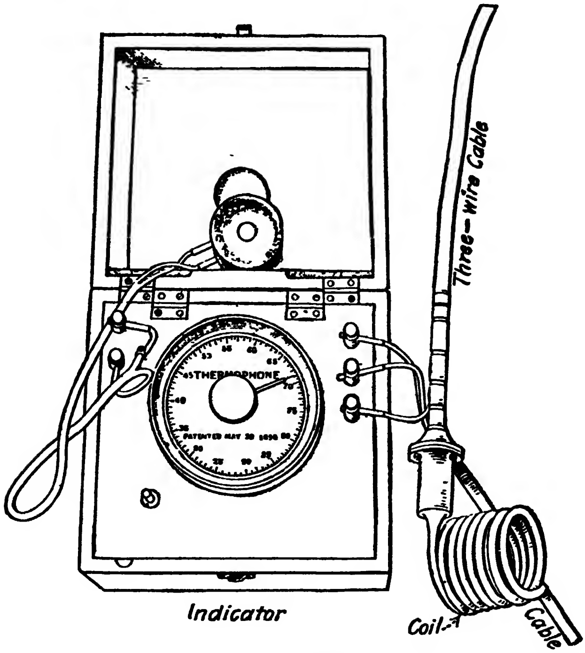


FIG. 347.

Circulation Due to Temperature. Because water is at its greatest density at 4° C. (39.2° F.) it follows that fluctuations in surface temperatures of lakes and ponds cause vertical circulation. As the surface water becomes cooler in winter, it becomes denser and sinks to the bottom. This overturn continues until a period of fairly stable equilibrium, known as winter stagnation, is reached. In the spring, the surface water begins to grow warmer and, until it reaches 4° C. grows denser and sinks. Therefore there is a period of circulation until all the water has reached the temperature of maximum density. These changes are known as the fall and spring circulations or overturns, respectively.

Lakes are divided into three types according to their surface temperatures and into three orders according to their bottom temperatures. The three types are *polar*, *temperate* and *tropical*. In lakes of the polar type, surface temperature is never above that of maximum density; in lakes of the temperate

type it is sometimes below and sometimes above it; in lakes of the tropical type it is never below that point. The three orders may be defined as follows: Lakes of the first order have bottom temperatures which are practically constant at or very near maximum density; lakes of the second order have bottom temperatures which undergo annual fluctuations, but which are never very far from maximum density; lakes of the third order have bottom temperatures which are seldom very far from the surface temperatures.

Table 245. Circulation Periods of Lakes
(Whipple)

Kind of lake	Polar type	Temperate type	Tropical type
First order......	One circulation period possible in summer, but generally none.	Two circulation periods possible, in spring and fall, but generally none.	One circulation period, possible in winter, but generally none.
Second order... .	One circulation period, in summer.	Two circulation periods, in spring and autumn.	One circulation period, in winter.
Third order......	Circulation at all seasons, except when surface is frozen.	Circulation at all seasons, except when surface is frozen.	Circulation at all seasons.

Wind Currents. Wind produces important horizontal currents. The velocity of surface water in Lake Erie is about 5 per cent. of the air velocity. Ackermann showed that the surface water of Owasco Lake (N. Y.) had a velocity of 3 per cent. of a wind velocity of 5 mi. an hour, and 1 per cent. of a wind velocity of 30 mi. an hour. After wind-driven water strikes the shore, it runs back below the surface. This is called an *undertow current.* Between the surface and the undertow currents there is a slow-motion zone called the shearing plane.

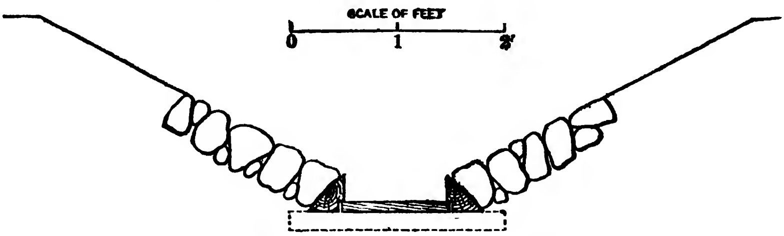
Removal of Color. Storage removes color because of the bleaching action of sunlight and the coagulation and precipitation of colored vegetable matter held in colloidal solution. The sun's rays are rapidly absorbed by water and their bleaching action is greatest at the surface. In clear waters there is little bleaching action below a depth of 5 ft. and in turbid waters the effect of sunlight may be felt only a few inches. Wachusett* records from 1907 to 1914 show that rate of reduction of color at bottom of reservoir was 30 per cent. less than at surface. In Ashland and Hopkinton reservoirs (Mass.) bottom decolorization is about 50 per cent. of that at surface. In some waters coloring matter is in colloidal solution, for example Dismal Swamp. Such coloring matter is often reduced quite suddenly, by the coagulation of fine particles and their rapid precipitation. *Sunlight* is not indispensable for the coagulation of colloidal color. *Iron* is a component part of chlorophyll and its derivatives, from which the coloring matter in water is derived. As the coloring matter is decomposed in the presence of oxygen, the iron is changed to its insoluble ferric state and is precipitated on the bottom of the reservoir.

* Metropolitan Waterworks, Boston.

During the warmer months, the bottom water in deep reservoirs becomes robbed of its oxygen, the iron is reduced to the soluble ferrous state and enters into combination with the organic matter. When this compound reaches the surface it oxidizes and increases the color of the water. Exposure to light, oxygen and bacteria bring about a decomposition of the coloring matter and an oxidation and precipitation of the iron.

Hydrogen sulphide is frequently produced by putrefaction of organic matter in the stagnant bottom water in a reservoir. This forms ferrous sulphide with the iron and imparts a black color to the water. Water from the bottoms of reservoirs containing excessive CO_2 or organic acids is apt to corrode lead pipes rapidly, making necessary the addition of lime and sometimes aeration.

Swamp drainage is an effective method of color prevention. Study of a watershed frequently determines certain swampy areas which are the principal sources of color. Desmond FitzGerald estimated that by the expenditure for drainage of $12,000, the equivalent of $44 per acre of swamp, or $8.57 per million gals. of storage capacity, the color of the water supplied by the Ashland reservoir (Metropolitan Waterworks, Boston) could be reduced from 56 to 45.



SECTION OF SWAMP DRAINAGE DITCH

FIG. 348.

(T A. S. C. E, Vol. 44, 1900, p 176)

F. P. Stearns used a simple form of *swamp drainage ditch* shown in Fig. 348. In connection with the Metropolitan Water works (Boston) 1069 acres of swamp were drained by 144,558 ft. of ditches. The cost, including other work and engineering expenses, also surveys of land not at that time drained, amounted to $93 per acre of swamp and 69 cts. per lin. ft. of ditch. Labor and materials alone cost as low as 23 cts. per lin. ft.

Soil Stripping. Frequently, especially in Massachusetts, bottoms and sides of reservoirs are stripped of all growing vegetation and surface soil, and all deposits of muck are covered with a layer of clean sand. Waters stored in stripped reservoirs are greatly improved in color and in odor (due to organisms) especially during the first few years. Ralph H. Stearns (Jour. N. E. W. W. Ass'n, 1915) has studied the decolorization of water in certain stripped and unstripped reservoirs in Massachusetts. See Table 246, also E. N., May 6 and Aug. 12, 1915, articles by Hazen and Whipple and F. P. Stearns. More data exist for stripped than for unstripped reservoirs. Ultimately the character of the bottom and sides of a reservoir will be controlled almost entirely by the accumulation of ooze precipitated from the water. (See page 107.)

Table 246. Decolorization of Water by Storage
(R. H. STEARNS)

Name of reservoir and supply (all in Massachusetts)	Years used in, average (inclusive)	No of years used	Area of water-shed, sq mi.	Storage capacity		*Yield per sq. mile, mil. gals	Length of storage, days	Average depth of res'r, ft.	Color‡ of influent water	Color‡ of water at dam	Per cent. color remaining	Per cent. color reduction
				Total	Per sq. mi.							
		Stripped reservoirs										
1. Wachusett	1908–1914	7½	118.0	65,000	550.0	0.894	615	48	43.5	14.7	34 0	66.0
2. Wachusett	1907	¼	118 0	38,600	327.0	0.920	355	39	45 0	23.0	51.0	49.0
3. Wachusett	1906	1	118.0	34,500	292.0	1 043	280	38	46.0	24.0	52.0	48.0
4. Hopkinton, Sudbury	1895–1897, 1899–1903	8	5.86	1,520	260.0	1.111	234	25	113.0	60.0	53.0	47.0
5. Ashland, Sudbury	1891–1897, 1899–1903	12	6.43	1,416	220.0	1.066	206	26	100.0	65.0	65.0	35.0
6. Reservoir No. 3, Sudbury	1891–1896	6	27.0	1,180	45 0	1 012	45	15	84.0	69.0	82.0	18.0
7. Reservoir No. 2, Sudbury	1900–1903, 1905–1914	14	47 0	530	11.3	0 863	13	12	76.0	69.0	91.0	9.0
8. Fall Brook, Leominster	1898–1903, 1905–1913	15	1.26	386	306 0	1.075	285	14	36.0	15.5	43.0	57.0
9. Hobbs' Brook, Cambridge	1900–1913, 1905–1914	14	7.25	2,097	289.0	0 863	335	14	59.0	18.0	30.5	69.5
		Unstripped reservoirs										
10. Borden Brook, Springfield	1910–1914	5	8.0	2,500	312.0	0.925	338	36	75.0	57.0	76.0	24.0
11. † Milham Brook, Marlborough	1901–1903, 1905–1913	12	3.56	315	88 0	0 853	103	15	47.0	45.0	96.0	4.0

* Based on Wachusett or Subdury records for the years given.
† Stripped to 10 ft. below flow line, but oxygen becomes exhausted in deep portion.
‡ Parts per million, see page. 788

Organic matter **which has the greatest effect upon the quality of impounded waters is derived from grass, weeds, trees and other vegetation on the reservoir site. They should be removed by cutting and burning just before the reservoir is filled. To reduce growths of aquatic weeds and filamentous algæ, the shores of the reservoir from 2 to 5 ft. vertically above the high-water mark and 10 to 20 ft. or more below, according to range of surface fluctation, should be grubbed of stumps and roots. Below this zone, stumps should be pulled or cut close to ground. It is well to remove all deciduous trees which are so near the reservoir that leaves would fall therein, or to provide a screen of arbor vitæ or similar evergreens. Stripping undoubtedly reduces growths of organisms, but it is the result of experience that it does not entirely or uniformly eliminate unpleasant or offensive odors. This is shown by Massachusetts experience. Stripping alone cannot be relied upon to produce a stored water which will be satisfactory at all times, in taste, odor or color. Modern standards call for a color less than 10 p.p.m. and no unpleasant odor at any time. Such a result can be obtained only by other methods of purification, such as filtration. Where filtration is necessary, the expense of complete soil stripping is seldom warranted. After the first few years, no considerable portion of the cost of stripping can be saved through a resultant reduction in the cost of filter operation.**

Effect of Storage on Bacteria. Storage reduces the total numbers of bacteria as well as B. coli. The latter are reduced to a proportionally greater extent than are the ordinary water forms. Storage also devitalizes disease

Table 247. Vitality of the Typhoid Bacillus
(Houston)

Experiment	Initial number of typhoid bacilli per c.c. of the infected *raw* river water	Number of typhoid bacilli per c c. of the infected *raw* river water, after storage in the laboratory for following periods, weeks					Number of weeks required to effect the destruction of the typhoid bacillus in 100 c c. of the infected *raw* river water
		One	Two	Three	Four	Five	
1 T.......	40	0	.....	..			Five
2 L.......	40	0	....	..			Five
5 L.......	170,000	53	2	0			Five
15 N. R....	525,000	29	3	0	. .	.	Five
3 N. R....	40	0	.. .	... ..	.	..	Six
4 T.......	170,000	9	2	0	..		Six
							Six
6 N. R...	170,000	40	2	0	...		
8 L.......	470,000	850	11	7	2	0	Seven
9 N. R....	470,000	1,430	14	7	0	. . .	Seven
14 L.......	525,000	32	2	0	..	... .	Seven
18 N. R....	475,000	30	3	0	.	. . .	Seven
7 T.......	470,000	480	31	5	0	..	Eight
10 T.......	8,000,000	3,000	30	4	0		Eight
11 L.......	8,000,000	2,900	29	5	0	.	Eight
13 T.......	525,000	12	1	0	......	. ....	Eight
17 L.......	475,000	80	11	2	0	.....	Eight
12 N. R....	8,000,000	400	22	2	0	......	Nine
16 T.......	475,000	210	12	2	1	0	Nine

T. = Thames, L. = Lee, N. R. = New River.

Table 248. Bacteria in Thames River Water Before and After Storage, 1907–08
(HOUSTON)

	Numbers of bacteria per c.c.			Per cent. of samples testing positive for B. coli				
	On gelatin, 20° to 22° C.	On agar, 37° C.	On bile-salt agar 37° C.	100 c.c.	10 c.c.	1 c.c.	0.1 c.c.	0.01 c.c.
River Thames..	4465	280.0	41.0	99.9	97.7	83.1	48.3	10.1
Chelsea stored water..	208	44.0	5.0	57.2	32.5	13.4	3.3	1.1
Percentage reduction.......	95.3	84.3	87.8	....	....	....	.. .	. ..

Table 249. Average Efficiency of Washington Water Purification System

Reservoir outlet	1907	1908	1909	1910	1911	1912	Average for 6 yrs.
			Turbidity				
Dalecarlia..............	46	53	50	30	18	59	43
Georgetown.............	37	45	32	29	16	23	30
McMillan Park.........	29	31	22	18	10	13	22
Filtered water..........	2	2	1	1	0	0	1
			Bacteria				
Dalecarlia..............	1,940	2,700	1,950	13,850	3,370	6,000	4,970
Georgetown........ . ..	1,680	2,940	950	10,850	2,080	2,600	3,350
McMillan Park..	635	1,250	390	6,820	1,390	1,100	1,930
Filtered water . .	31	55	21	143	38	35	54

Table 250. Influence of Temperature on Survival of Typhoid Bacteria
(HOUSTON)

Temperature	No. of weeks and No of bacteria								
	1	2	3	4	5	6	7	8	9
0° C.	47,766	980.0	65.0	34.0	3.0	3.0	2 0	1.0	0.0
5° C.	14,894	26.0	6.0	3 0	0.3	0.1	0 0		.. ...
10° C.	69	14.0	3 0	0.3	0.0	.. ...	. .		. ..
18° C.	39	3.0	0 4	0 0	.			.	..
27° C.	19	0.1	0.0	...	.				.
37° C.	5	0.0	. ....	.. .					.

Initial number of cultivated typhoid bacteria added = 103,300 per c.c , Thames river water.

germs, particularly typhoid and cholera bacteria. Typhoid fever bacteria die out more rapidly at higher than at freezing temperatures. They die out in ice in spite of cold rather than because of it.

Effect of Storage on Microscopic Organisms. In order to prevent growths of organisms water from the drainage area should be delivered quickly and stored in a reservoir which will not add to the organic content. It should not be allowed to stand for any considerable length of time in contact with organic matter. In deep reservoirs, little life exists below the transition zone.*

* Transition Zone, sometimes called Thermocline or Discontinuity Layer—German: Sprungschicht—is the relatively thin layer between the upper and lower layers, where the temperature changes rapidly with the depth.

However, Crenothrix and fungi may develop in stagnant zones. Most growths of organisms occur near the surface where light is abundant. Vertical circulation causes distribution of algæ as well as of food for their growth. Under the influence of sunlight in the upper layers, algæ grow readily, and are held at the surface by the gases evolved during growth. *Growths of microscopic organisms* vary with the seasons as do flowering plants. In northeastern U. S. diatoms exhibit a spring and winter growth. The former is usually at its hight in May, the latter in September. Chlorophyceæ (green algæ) reach their maximum in summer (June and July) and are followed by Cyanophyceæ, which reach their maximum in October. The most troublesome forms of Cyanophyceæ, *e.g.*, Anabæna, rarely develop at a temperature below 60 to 70° F., although growths may linger until frozen into the ice. Seasonal distribution of Protozoa is very variable. Rotifera are most numerous in summer and growths of Crustaceæ, as a rule, reach their maximum in the spring.

Storage of ground waters should invariably be in the dark to prevent growths of microscopic organisms, especially diatoms. Of these Asterionella is the most troublesome. Storage may not prevent the growth of fungi and certain Protozoa which do not require light for their existence.

Storage of Filtered Waters. Filtered waters, especially those high in mineral matter and carbon dioxide, have to be stored like ground waters. Some very soft waters from upland sources may be stored in open reservoirs with impunity. Concrete masonry makes the most satisfactory cover as it excludes both light and heat. Floating roofs of wood have not been generally successful. Wooden roofs deteriorate rapidly because of dampness. Roofs of cement or asbestos slabs supported by steel framework may be advisable where the effect of temperature on the water may be neglected.

CHAPTER XXXIII

SEDIMENTATION

Plain subsidence or sedimentation serves to remove the subsidable silt and clay. It is the cheapest method of removing those particles which settle out in a moderately short time and which would rapidly clog a filter. Subsidence is effected in open basins with concrete floors or in impounding reservoirs which are designed to hold from a few hours' to several days' supply. In the *intermittent* system a basin is filled, allowed to stand and then is drained. In the *continuous* system water flows through the basin constantly. Cleaning is usually accomplished hydraulically by opening gates and flushing out the sediment. Bottoms of well-designed subsiding basins usually slope to drains or openings. Hose streams are frequently used to move the sediment to the drains. A very few plants employ hydraulic dredges. Self-cleaning basins are not practicable because of the large size of some sediment particles, although sand ejectors have been built in the bottoms of grit chambers (Cleveland).

Theoretical Considerations. The theory of subsidence has been elucidated by Hazen and others. (T. A. S. C. E., **53**, 1904, 45.) Every particle in suspension tends to move downward at a velocity depending upon its size, weight, shape and upon the viscosity of the water. Spherical particles settle more rapidly than irregular particles of the same specific gravity. Particles in colloidal solution are so fine that they are not acted upon by gravity unless coalesced into groups. Water containing sediment and free from colloidal matter (*e.g.*, clay) in a perfectly quiet basin will clear most rapidly at the top; the coarse particles will go down faster and there will be a progressive clearing downward. Most minerals suspended in water have a specific gravity of from 2.1 to 2.9. The sp. gr. of quartzite sand is 2.65.

Table 251. Velocities at which Particles of Sand and Silt will Subside in Still Water

(Hazen, Wiley and Others)

Diam. of particles, in mm.	Hydraulic value, in mm. per sec. at 10° C. = 50° F.	Diam. of particles, in mm.	Hydraulic value, in mm per sec. at 10° C. = 50° F.
1.00	100*	0.03	1.3†
0.80	83*	0.02	0.62‡
0.60	63*	0.015	0.35‡
0.50	53*	0.010	0.154‡
0.40	42*	0.008	0.098‡
0.30	32*	0.006	0.055‡
0.20	21*	0.005	0.0385‡
0.15	15*	0.004	0.0247‡
0.10	8*	0.003	0.0138‡
0.08	6†	0.002	0.0062‡
0.06	3.8†	0.0015	0.0035‡
0.05	2.9†	0.001	0.00154‡
0.04	2.1†	0.0001	0.0000154‡

* Experiments by Hazen. † Interpolated from connecting curves. ‡ Wiley's formula.

Table 252. Grades of Suspended Particles in Water

Kind of material	Diam. of particles, in mm.	Kind of material	Diam. of particles, in mm.
Coarse gravel..	2 upward	Silt................	0.05–0.01
Fine gravel....	2–1	Fine silt............	0.01–0.005
Coarse sand...	1–0.5	Clay................	0.01–0.001
Medium sand..	0.5–0.25	Fine clay...........	0.0010–0.0001
Fine sand.....	0.25–0.1	Colloidal clay......	Finer than 0.0001
Very fine sand.	0.1–0.05		

Basin Types. When

t = time required for a particle of sediment to fall from surface to bottom of water in basin, water meanwhile being absolutely still;

a = time of sedimentation in case action is intermittent (in case of continuous operation, let a be quotient obtained by dividing capacity of basin by quantity of water entering or leaving it during each unit of time);

n = number of basins, in case several basins are used successively;

x = proportion of sediment remaining at end of process, amount at beginning being taken as unity;

the values of x for various values of $\frac{a}{t}$ are plotted in Fig. 349 as line A. This is the theoretical maximum and cannot be reached in practice because suspended matter does not subside in the manner indicated. Its sub-

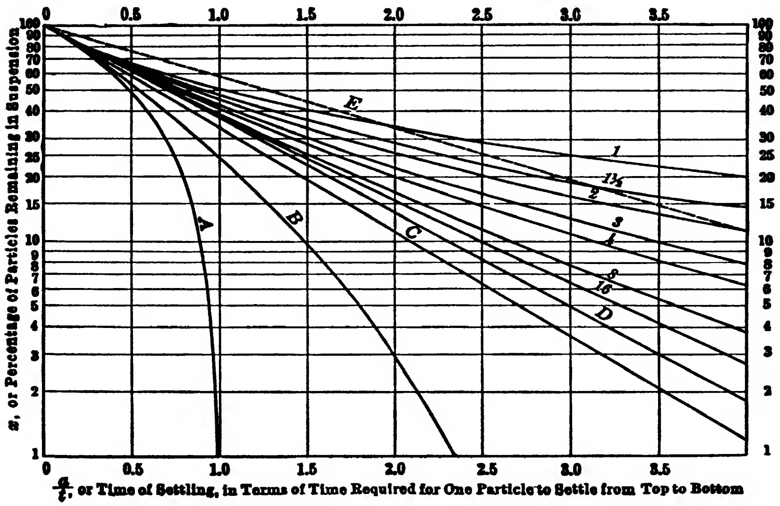


Fig. 349.
(T. A. S. C. E., Vol. 53, 1904, p. 47.)

sidence is interfered with by the vortex motion produced by the kinetic energy of the entering water, by the action of the wind, by convection currents produced by temperature, and by mixing produced by currents caused by the shape of the basin.

Values of $\frac{a}{t}$ necessary to secure the removal of ½, ¾ and ⅞ of the particles of a given size, with basins of different arrangements, and without regard to

excessive bottom velocities or the favorable effect of contact with the already subsided particles, are given in Table 253.

Table 253. Comparison of Different Arrangements of Settling Basins
(HAZEN, T. A. S. C. E., VOL. 53, 1904)

Description of basins	Line in Fig. 349	Values of $\frac{a}{t}$		
		One-half removed	Three-fourths removed	Seven-eighths removed
Theoretical maximum (cannot be reached) ...	A	0.50	0.75	0.875
Surface skimming, Rochner-Roth system... .. .	B	0.54	0.98	1.37
Intermittent basins, reckoned on time of service only..................................	C	0.63	1.26	1.89
Continuous basin, theoretical limit............	D	0.69	1.38	2.08
Close approximation to ditto.................	16	0.71	1.45	2.23
Very well baffled basin......................	8	0.73	1.52	2.37
Good baffling................................	4	0.76	1.66	2.75
Two basins tandem........................	2	0.82	2.00	3.70
One long basin, well controlled...........	1½	0.90	2.34	4.50
Intermittent basin, in service half time....	E	1 26	2 50	3 80
One basin, continuous...................	1	1 00	3 00	7.00

Lines corresponding to basins of different arrangements noted in Table 253 are shown in Fig. 349.

The diameter of particles in millimeters, such that 75 per cent. will be removed with continuance of operation, may be computed, according to Hazen, by the formula:

$$d = 0.0027f\sqrt{\frac{\text{million gallons daily}}{\text{area of basin in acres}}}\sqrt{\frac{60}{t+10}}$$

"in which f is a factor depending upon the arrangement of basins and baffling. Use 1.73 for a basin with one inlet and one outlet well separated; 1.41 for two basins through which the water passes successively; 1.22 for a well-baffled basin or other specially good arrangement. $f = 1.00$ is a theoretical limit not reached in practice. In the last term t is temperature in degrees Fahrenheit. For comparisons use $t° = 50$ in all cases. The rule does not apply for separations above 0.05 mm. It is not precise, but it affords a convenient basis for comparing various sedimentation and coagulating basins."

Effect of Temperature. Because of greater viscosity of water at low temperatures, a particle of silt will settle twice as fast at a temperature of 23° C. (74° F.) as at 0° C. (32° F.). The rate of settling at different temperatures varies as $\frac{t°\text{ F.}+10}{60}$. Annual average temperature of water in lakes, ponds and open reservoirs in northern U. S. is about 10° C. = 50° F.

Practical Considerations. Quantities of solids carried in suspension by several American rivers are given in Table 25. Results obtained in various subsiding and coagulating basins are given in Table 254. Experiments at New Orleans showed that from Mississippi river water, containing an average of 650 p.p.m. of suspended matter and an average turbidity of 600 p.p.m., the amounts of suspended matter given in Table 255 would be deposited.

Table 254. Subsiding and Coagulating Basins

(Hazen, Trans. Amer. Soc. C. E., Vol. 53, 1904, p. 70)

Basin	Area, sq. ft.	Average, depth, ft.	Approximate horizontal course, ft.	Ratio of horizontal course to depth	Quantity treated daily, gals.	Hours of storage	Gals. per sq ft. daily
East Jersey	5,400	43.0	130	3	32,000,000	1 30	5,920
Jewell, Pittsburgh	131	6.7	....	......	250,000	0.63	1,910
Warren, Pittsburgh	176	10.0	.... .	....	300,000	1 05	1,700
Ithaca	4,700	11.4	240	23	3,000,000	3.20	638
Watertown	9,880	13.6	270	20	6,000,000	4.00	610
World's Fair, Chicago, 1893 sewage	3,136	40.0	....	.. . .	2,400,000	9 00	768
Albany	220,000	8.5	400	47	17,000,000	20 00	77
Experimental sand filters Pittsburgh	670	6 8	45	7	34,000	24.0	51
St Louis	1,600,000	14 0	750	54	80,000,000	50 0	50
Kansas City	300,000	20 0	1,700	85	15,000,000	72 0	50
Washington	6,690,000	14.0	8,000	570	75,000,000	225.0	11

Basin	Horizontal velocity, mm. per sec.	Mm. in depth of water removed per sec.	Type of basin, Fig. 349	Value of a/t for 75 per cent. removal	Hydraulic value of smallest particles removed*	Diam. of smallest particles removed, mm.
East Jersey	8 0	2 78	Special	2 00	5.56	0.077
Jewell, Pittsburgh	....	0.90	1	3 00	2 70	0 047
Warren, Pittsburgh	.	0 80	2	2 00	1 60	0 034
Ithaca	6 0	0.30	4	1.67	0 50	0 018
Watertown	6 0	0.286	4	1.67	0 48	0.018
World's Fair, Chicago, 1893 sewage	....	0.360	B	1.00	0 35	0.015
Albany	2.0	0.0363	1½	2.33	0 084	0.007
Experimental sand filters Pittsburgh	0 2	0.0240	1	3.00	0 072	0 007
St. Louis	1 0	0.0235	1	3 00	0 070	0 007
Kansas City	2 0	0.0235	4	1.67	0.039	0.005
Washington	3 0	0.0053	4	1 67	0.009	0 003

* See definition on p. 741.

Table 255. Estimated Amounts of Suspended Matter which Would be Deposited from Mississippi River Water

Period of subsidence, hours	Suspended matter, parts per million	Percentage removed		Mud removed	
				Per million gallons	
		Silica turbidity	Suspended matter	Cubic yards	Tons
12	215	19	33	2.28	2.48
24	290	28	45	3.07	3.35
48	350	37	54	3.71	4.05
72	385	42	59	4.08	4.45

Sp. gr. of mud at New Orleans = 1.3; weight of mud per cu. ft. = 81 lb.

Results. As a rule, well-baffled basins having a capacity equal to 6 hrs.' flow will remove particles larger than 0.02 mm. in diameter. Basins having a capacity equal to 24 hrs.' flow will remove particles larger than 0.007 mm. At Washington, D. C., in a succession of three reservoirs (see Table 249, p. 682) holding a week's supply, particles larger than 0.003 are removed. Removal of the bulk of the suspended matter is usually all that is necessary as a preliminary to filtration. Suspended matter in *clay-bearing streams* cannot be completely removed even after weeks of subsidence. Intermittent basins possess no advantage over properly designed continuous basins. Allowing for the time out of service, they do only slightly more efficient work, and furthermore deliver the water at a lower elevation. Ordinary baffling raises the efficiency of continuous above that of intermittent basins. Horizontal basins should have a length not exceeding one and one-half times their width. Greatest efficiency is obtained in a high vertical basin with the entering water well distributed at the bottom and the effluent skimmed from many points at the surface; but this type is impracticable for

Table 256. Variations of Turbidity and Temperature of Water in Subsiding Basins at Various Depths*

(Source, S. B. 1, New Orleans Experiments)

Date 1901	Hour	River water		Depth, ft.	Inlet end		Outlet end	
		Silica† turbidity	Temperature, degrees Fahr		Silica turbidity	Temperature, degrees Fahr.	Silica turbidity	Temperature, degrees Fahr.
Mar. 26	10	900	52 1	0	10	62 6	20	64.4
				2	12	61.7	25	61.7
				4	190	56.3	220	56.3
				6	380	56.3	400	56.3
				8	480	55.4	440	55.4
Mar. 27	9	950	54.6	0	40	64.4	80	63.5
				2	180	58.1	180	58.1
				4	360	55.4	360	56.3
				6	500	55.4	550	55.0
				8	625	54.6	600	55.0
Mar. 28	10	975	53.2	0	100	....	100	....
				2	270	....	270	....
				4	400	....	380	....
				6	440	....	480	....
				8	600	....	600	....

* T. A. S. C. E., Vol. 53, 1904, p. 76. † Parts per million.

large plants. Where the entering water is colder than the air, the water in the basin will stratify. (See Table 256.)

Because efficiency of basins depends in part upon area of bottom surface upon which sediment can be deposited, and because best results are obtained when the incoming water can be brought into contact with the sediment and kept from mixing with the partially clarified water, it is well to divide horizontal basins into consecutive compartments by baffles. Of these,

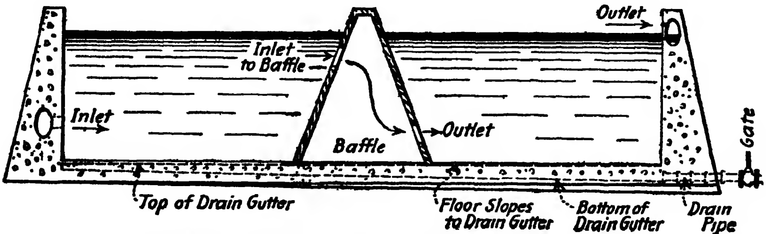


FIG. 350.—Subsiding basin with A-frame baffle.

the *A-frame baffle,* shown in Fig. 350, is the most useful. This baffle receives the water from the upstream compartment through a number of openings near the surface and discharges it near the bottom of the basin through corresponding openings. They are usually built of concrete slabs supported by A-frames of steel or reinforced concrete, or of plank supported by steel frames. Bottom velocities affect the deposition of sediment. Sediment once deposited is not so easily conveyed by the water as before.

Chemical Formulæ Commonly Used in Water Purification

Substance	Formula
1. Acids	
Acetic	CH_3COOH
Hydrochloric	HCl
Nitric	HNO_3
Oxalic	$H_2C_2O_4$
Sulphuric	H_2SO_4
2. Salts	
Alcohol	CH_3OH
Aluminum hydrate	$Al_2(OH)_6$
sulfate	$Al_2(SO_4)_3$
Ammonium chloride	NH_4Cl
ferrous sulfate	$(NH_4)_2Fe(SO_4)_2, 6H_2O$
oxalate	$(NH_4)_2C_2O_4, 2H_2O$
persulfate	$(NH_4)_2S_2O_8$
Barium carbonate	$BaCO_3$
chloride	$BaCl_2$
sulfate	$BaSO_4$
Calcium carbonate (calcite)	$CaCO_3$
chloride	$CaCl_2$
oxide	CaO
sulfate	$CaSO_4, 2H_2O$
Chloroform	$CHCl_3$
Cobaltous chloride	$CoCl_2, 6H_2O$
Copper sulfate	$CuSO_4$
Ferric chloride	$FeCl_3$
oxide	Fe_2O_3
sulfate	$Fe_2(SO_4)_3$
Ferrous carbonate	$FeCO_3$
oxide	FeO
sulfate	$FeSO_4$
Hydrogen peroxide	H_2O_2
Lead acetate	$Pb(C_2H_3O_2)_2$

Substance	Formula
Magnesium carbonate	$MgCO_3$
chloride	$MgCl_2$
sulfate	$MgSO_4$
Manganous sulfate	$MnSO_4$
Mercuric chloride	$HgCl_2$
Phenol	C_6H_5OH
Potassium bichromate	$K_2Cr_2O_7$
carbonate	K_2CO_3
chloride	KCl
chromate	K_2CrO_4
hydrate	KOH
hydrogen sulfate	$KHSO_4$
iodide	KI
nitrate	KNO_3
nitrite	KNO_2
oxalate	$K_2C_2O_4, 2H_2O$
permanganate	K_2MnO_4
platinic chloride	K_2PtCl_6
sulfate	K_2SO_4
sulphocyanide	$KSCN$
Silica	SiO_2
Silver chromate	Ag_2CrO_4
nitrate	$AgNO_3$
sulfate	Ag_2SO_4
Sodium bismuthate	$NaBiO_3$
Sodium carbonate	Na_2CO_3
chloride	$NaCl$
nitrate	$NaNO_3$
nitrite	$NaNO_2$
sulfate	Na_2SO_4
thiosulfate	$Na_2S_2O_3$

CHAPTER XXXIV

AERATION* AND CHEMICAL TREATMENT

Aeration consists in bringing water into intimate contact with air in order to introduce oxygen for the oxidation of iron or organic matter, and for washing out gases and volatile odors. Water sprayed in fine streams from varying hights absorbs oxygen rapidly. For the solubility of oxygen, see p. 643.

Table 257. Oxygen Absorbed by Water Sprayed from Various Hights (OESTEN)

Hight of fall		Oxygen absorbed in parts per million
Centimeters	Inches	
10	4	1 21
25	10	1 79
50	20	2 52
100	40	6 50
200	80	7 33

Table 258. Carbonic Acid Left in Solution after Aeration by Exposure in Drops (G. C. AND M. C. WHIPPLE)

	Carbonic acid (parts per million)			
At the start	5 0	10.0	25 0	50 0
After 0.5 sec	4 1	6.9	13 8	23.4
After 1 sec	3.5	5 3	9 3	14.0
After 2 sec	3 0	4 1	6 2	8.5
After 5 sec	2 5	3.0	3 8	4.5
After 15 sec	2 1	2 1	2 1	2.1

Table 259. Hydrogen Sulfide Left in Solution after Aeration by Exposure in Drops (G. C. AND M. C. WHIPPLE)

Time	Sulfureted hydrogen (parts per million)	Odor
At start	15.2	Faint
After 1 sec	10.2	Very faint
After 1.5 sec	5.0	Very faint
After 2.0 sec	2.6	None

The removal of dissolved gases and odors is difficult. Aeration may be accomplished by sprays, cascades or fountains. Small streams are better than large. Good dispersion is produced where the water revolves in the pipe before it issues from nozzle; various spray nozzles† on the market embody this principle. Cascades economize head, and, when the water is distributed in thin sheets, are very efficient. Superimposed pans with alternate per-

* Aeration has successfully removed odors from waters at Newark, N. J.; Grassy Sprain res., Yonkers, N. Y.; Whiting St. res., Holyoke, Mass.; Ludlow res., Springfield, Mass. It has been undertaken on large scale for New York City's Catskill supply. through nozzles causing exposure of water in spray for a few seconds under at least 16-ft. head. (See p. 260.) Such aeration will not only oxygenate, but reduce free carbonic acid from about 20 to 5 p. p. m. and remove probably three-fourths of odors of growth and decomposition.

† See pages 259 and 260.

forated and plain sections, Fig. 351, combine the advantages of spray and cascade aerators.

Interchange of gases may be accelerated by contact, as accomplished by a tower or trickler filled with coke, slag or stone, (several European plants use brick; also wooden slats) such as used in deferrization plants. The contact principle applies in all filters, especially where waters containing heavy growths of algæ, much organic matter or colloidal iron make such treatment necessary. Oxygen is condensed in the films on the sand during the idle period and given off during the working period. (Dunbar, Zeit. für Hygiene, **22**, 68 (1896); Ludlow Filter at Springfield, Mass., Jour. N. E. W. W. Ass'n, **21**, 279 (1907).

FIG. 351.—Aerator at Winchester, Ky., pumping station. (Wheeler.)

Coagulation. Water containing less than 30 p.p.m. of turbidity or less than 20 p.p.m. of color can be purified by filtration through sand without coagulation. Filtration alone cannot completely remove colloidal clay, organic matter, or dissolved vegetable color. These are in colloidal solution and must be coagulated before filtration.

Theory of colloids helps to explain many phenomena of water purification. The old ideas of Graham that colloids (*e.g.*, glue) were those substances which would not dialize through parchment, and crystalloids (*e.g.*, salt) were those which would, has been abandoned. Colloids are simply matter in a finely divided or highly dispersed condition, yet not so finely divided as to form true molecular solutions. In order to be dispersed in colloidal form, a substance must be so finely divided that it will remain in suspension indefinitely. A lump of clay-schist sinks rapidly, but when disintegrated by water the clay particles of which it is composed remain in suspension indefinitely, like those in the Mississippi River water. Fine particles do not follow the law of coarse subsiding particles, and because of molecular movements (called Brownian) which are negligible for coarse particles, remain suspended. Particles of vapor and dust in the air are often in the colloidal state. Gold, paper, clay, iron and other substances may be so finely divided as to remain suspended in pure water indefinitely.

The force which holds these very fine particles in suspension is a property of their surfaces, a film produced by the selective powers of absorption which these surfaces possess, *e.g.*, the condensation of oxygen on the surfaces of the sand grains in a filter; or the electrical charge which causes the unprecipitated particles of water in the atmosphere to repel one another, thus preventing their coalescence and consequent precipitation.

Particles in the colloidal state are smaller than 0.0001 mm. in diameter and in some cases no larger than 0.000,006 mm. The enormous increase in surface area due to division of matter, greatly augments the capacity of the

substance for absorption. This has an important bearing upon the power of aluminum and other hydrates used as water coagulants which, when precipitated in water, pass gradually through the colloidal into the true suspended state and in doing so offer, because of their small size, a correspondingly vast surface for the attraction of suspended matter such as clay, color and bacteria.

Colloidal solutions are of two kinds, those wherein the substances are dispersed in water (where water is the external phase) and those wherein water is within the expanded honeycomb structure of the substance (where water is the internal phase). Colloidal gold is an example of the first, gelatin of the second kind of solution. Where water is the external phase, and where the colloids are suspended electrically, the particles will migrate or move when an electric current is present, to the positive or negative poles, depending upon the preferential absorption of negative or positive electrical charges by the particles. Colloidal clay has a negative, aluminium hydrate a positive, charge. Colloids of like charges repel, while those of opposite charges attract one another. In Posen, Poland, the colloidal coloring matter in the deep well water and the colloidal iron in the aerated shallow well water mutually precipitate each other when the waters are mixed in the right proportions. Where water is the internal phase, some of the gelatin-like substances may pass in and out of solution by the expansion and contraction of the honeycomb-like structure of the substance. These colloids are "reversible." The existence of this structure may explain how certain vegetable matters, like vegetable albumens, may retard coagulation by preventing contact of the particles of coagulant, *i.e.*, they interpose their structure between the particles. Other colloids which, when thrown out of solution, cannot be taken up again without redivision are called "irreversible."

Electrolytes (dissolved salts which conduct current) tend to precipitate colloids. This is why coagulation in soft waters is more difficult than in hard. Any contact surface, particularly one covered with a film of already coagulated substance, favors coagulation, likewise agitation, mixing, friction, time or high temperature, all of which tend to bring the divided particles into contact.

Table 260. Principal Chemicals Used for Water Purification

Trade name	Strength	Formula of active ingredients	Price per 100 lb. delivered*
Basic sulphate of alumina.	16–23% Al_2O_3	$Al_2(SO_4)_3$, XH_2O† + XAl_2O_3	\$0.80–\$1.30
Ferrous sulphate . . .	95–100% $FeSO_4$, $7H_2O$	$FeSO_4$, $7H_2O$	0.60– 0.80
Copper sulphate.	99% $CuSO_4 + 5H_2O$	$CuSO_4$	4.50– 7.75
Quicklime . . .	75–99% CaO	CaO	0.25– 0.60
Hydrated lime. .	80–99% $Ca(OH)_2$	$Ca(OH)_2$	0.30– 0 65
Soda ash	58% Na_2O	Na_2CO_3	0.90– 1.75
Caustic soda.. ..	70–76% Na_2O	NaOH	2.00– 4.00
Barium carbonate.	98% $BaCO_3$	$BaCO_3$	2.00– 4.00
Bleaching powder	30–38% Cl	$CaCl_2$, $Ca(OCl_2)$	1.25– 2 50
Liquid chlorine .	100% Cl	Cl_2	7.00–10.00

Coagulation serves two purposes: (*a*) it gathers into groups or flocs particles which in a dispersed condition could not be removed by subsidence or filtration;

* These are normal ranges of prices. Prices have been abnormally high during the European war. Caustic soda, \$6.50; bleaching powder, \$15.00 (Mar., 1916). See E. N., Mar. 16, 1916, p. 530.

† X means uncertain number of molecules; the substance is a mixture.

(*b*) it forms honeycomb-like layers or films in the filter sand which allow water to pass at a high rate while at the same time retaining bacteria and suspended matter. The vast surface presented by the jelly-like compound which is formed by the addition of a coagulant to an alkaline water causes the absorption of color, organic matter and other substances, which exist in true or colloidal solution.

Basic sulfate of alumina, frequently though erroneously called alum, is the most commonly used coagulant. It is basic, *i.e.*, it contains more hydrate than will combine with sulfuric acid to form a neutral salt. Deficiency in acid (SO_3) averages about 5 per cent. Best sulfate of alumina contains 17.5 to 18.5 per cent. of Al_2O_3—which should be 3 per cent. or more in excess of the amount theoretically required for the sulfuric acid—not more than 0.5 per cent. of matter insoluble in cold distilled water, and not more than 0.2 per cent. of Fe. It should be furnished in lumps 0.5 in. to 2.0 in. in diameter for making solutions, or ground finer than 0.15 in. for feeding dry. Ordinary sulfate is made by dissolving the hydrate in sulfuric acid and evaporating to partial dryness. According to Frank E. Hale (Jour. Ind. and Eng. Chem., **6**, 1914, 632) it has the formula $2Al_2(SO_4)_3 : Al_2(SO_4)_2(OH)_2$—a mixture approximating 2 parts neutral sulfate and 1 part monobasic sulfate. Hoover (Columbus) mixes sulfuric acid and native aluminum hydrate (bauxite) and adds the chemical in liquid form. There is an impure sulfate called "alumino-ferric" which contains about 15 per cent. Al_2O_3 and nearly 1 per cent. of Fe'''.*

The theoretical maximum alkalinity computed as $CaCO_3$ that can react with 1 g.p.g. (grain per gallon) of ordinary 17.5 per cent. sulfate is 8.2 p.p.m., or 1 p.p.m. of alkalinity to 2.08 p.p.m. of sulfate. In practice the reduction in alkalinity may be as low as 60 per cent. of the theoretical or 5.0 p.p.m. per 1 g.p.g. of sulfate. The amount of CO_2 set free by the reaction with the alkali in water averages 3.4 p.p.m. per 1 g.p.g. (1 g.p.g. = 17.12 p.p.m.) See Table 306, page 788.

Colloidal clay absorbs some added coagulant and prevents it from reducing the computed amount of alkali. As color and turbidity decrease and as alkalinity and time increase, the theoretical reduction of alkalinity is approached and finally all the sulfate is replaced by carbonate, which latter hydrolyzes with the production of aluminum hydrate, $Al_2(OH)_6$. The reaction between sulfate of alumina and water is a time reaction and is retarded by cold. Combination of cold weather and low alkalinity frequently prevents complete coagulation of water until filtered, when "after-coagulation"† takes place. Usually an excess of alkalinity of at least 7 p.p.m. must be maintained to prevent after-coagulation and corrosion of water pipes. This excess may be secured by addition of lime or soda ash. The former alkali has the disadvantage of increasing the permanent hardness; the latter is more expensive and an excess may cause the coagulum to redissolve. Usually not much more than half the theoretical amount of soda ash required to produce the necessary alkali may be added safely; coagulum is soluble in excess of soda.

Sometimes compounds of iron and coloring matter exist in water; such

* F''' = ferric iron (trivalent); F'' = ferrous iron (bivalent).

†This is coagulation occurring after water is filtered.

cannot be completely decolorized by sulfate of alumina alone. The greater part of this residual color may be removed by treating the water with Cl before adding the sulfate. At Exeter, N. H., in 1914, the addition of an average of 0.5 p.p.m. of chlorine to the raw water with an average of 32.8 p.p.m. of sulfate of alumina, reduced the color from an average of 56 to an average of 6.6 p.p.m. During the previous year, without the use of chlorine, color could not be reduced below 27 parts, even with an increased dose of sulfate of alumina. This experience has been repeated at Belfast, Me., where an impounded reservoir water is treated, and at the Arlington Mills, Lawrence, Mass., where the Spicket river, which has had a color as high as 180 p.p.m., is treated. During the winter, in Belfast, it is impossible to reduce the color sufficiently or even completely coagulate the water without the addition of chlorine. At the Arlington Mills, the color was reduced during June, 1915, from an average of 83 p.p.m. to an average of 10, the dose of chemicals being 62 p.p.m. of sulfate of alumina, 10 of soda and 0.1 part of chlorine. Without chlorine, reduction of color to 10 p.p.m. or less was impossible.

Many *very highly colored waters*, like that of the Great Swamp, in the South (G. C. Whipple, Proc. Am. W. W. Ass'n (1911), pp. 261–276), which have a carbonate alkalinity of 9 and a color of 150 p.p.m., are often really acid with vegetable acids.* Addition to this water of up to 1.5 g.p.g. of sulfate of alumina produced no apparent reaction in 24 hrs. A further addition, up to 2.5 g.p.g., removed a large proportion of color, but addition of more than 4 g.p.g. could not reduce color below 25 p.p.m. To decolorize such waters completely, either the bulk of the precipitate must be increased by addition of more sulfate with alkali to react with it or by the addition of chlorine. An excess of sulfate of alumina lightens the color of natural water; an excess of alkali deepens it. The amount of color which can be removed by the same dose of sulfate of alumina varies greatly with the character of the coloring matter.

Ferrous sulfate in conjunction with lime is a very good coagulant for turbid, alkaline waters such as those of the Missouri and Ohio basins. Its formula is $FeSO_4,7H_2O$. The best form, known as "sugar of iron" or "sugar sulfate," is partially dehydrated and contains over "100 per cent." of sulfate calculated as $FeSO_4,7H_2O$ and less than 1 per cent. of foreign matter. Ferrous sulfate forms a coagulum of greater specific gravity and is considerably cheaper than sulfate of alumina. On the other hand, it must be used in conjunction with lime. The addition of two chemicals requires more, and more skilful, supervision than one. But the addition of lime where temporary hardness is high softens the water. Ferrous sulfate and lime cannot well be used with waters which are high in color or which are alternately turbid and colored. Neither can they well be used with soft waters, because any surplus lime would give the water a caustic alkalinity.

The reaction is complicated, but may be given as follows: The sulfate reacts with the bicarbonates in the water forming ferrous bicarbonate, viz: $FeSO_4 + CaCO_3H_2CO_3 = FeCO_3H_2CO_3 + CaSO_4$. This would oxidize and

* Some vegetable acids are weak and react alkaline with litmus, methyl orange, erythrosine and other indicators of relatively high acidity; they react acid to phenolphthalein and other indicators of low acidity.

precipitate too slowly for any practical use. Lime must be added to remove the CO_2 and complete the reaction,

$$FeCO_3H_2CO_3 + Ca(OH)_2 = Fe(OH)_2 + CaCO_3H_2CO_3.$$

The $Fe(OH)_2$ formed is a gelatinous precipitate similar in action to that produced by sulfate of alumina. It is, however, slightly soluble, but by oxidation is changed into the insoluble ferric hydrate, $Fe_2(OH)_6$, which is the coagulant desired. A slight excess of dissolved oxygen must be present to convert the ferrous to ferric hydrate. Iron bacteria are apt to grow in iron sulfate solutions, even those containing from 2000 to 4000 p.p.m. of Fe. $Fe_2(OH)_6$ can be precipitated directly by the addition of ferric sulfate ($Fe_2(SO_4)_3 + 9H_2O$), but the chemical is too costly and does not react with coloring matter so well as the basic sulfate of alumina. Sulfate of alumina may contain a small amount of ferric sulfate ("alumino-ferric") without reducing the efficiency of the coagulant for moderately hard turbid waters.

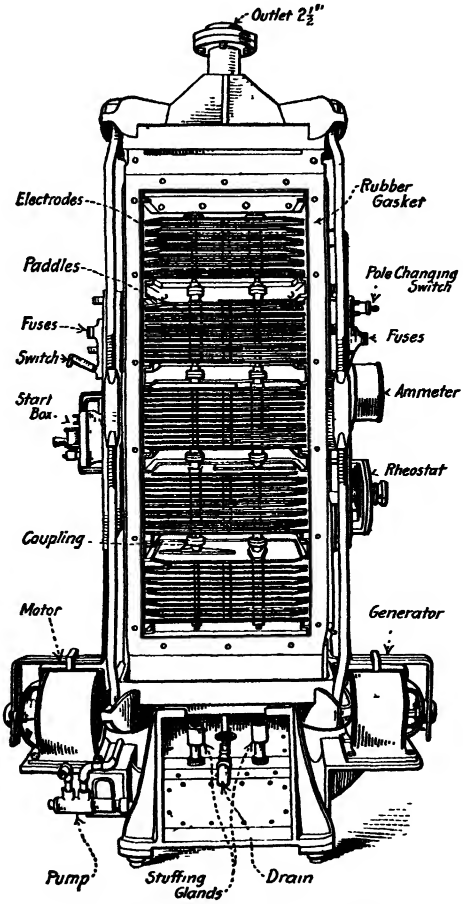


Fig. 352.—Apparatus for the electrolytical production of coagulant. Front view with door removed. Inlet not shown. (Electrolytic Purification Co.)

Aluminum and iron hydrates may be produced electrolytically by passing a current through plates immersed in the water to be treated. The compounds formed and the relative efficiencies of the coagulant so prepared and that formed by the addition of chemicals are given by George W. Fuller ("Water Purification at Louisville," 1898, Van Nostrand). The process has some advantages, the chief one being that it introduces no SO_4 ions* into the water. It was shown at Louisville, however, that considerable metal is lost by scaling, and that the accumulation of gases on the metal electrodes prevents the proper production of hydrate. Recent improvements in electrolytic ap-

* Ions are the constituents of an electrolyte which are *primarily* liberated or deposited from an electrolyte at an electrode. (Am. Handbook for Electrical Engineers, Wiley, 1915.) Thus H_2SO_4 electrolyses to H_2 and SO_4, H_2O to H and OH, their respective ions.

paratus are said to overcome these difficulties by means of mechanical devices to keep the electrodes clean and automatic electrical controls to prevent polarization and the ineffective application of current (Fig. 352). The process cannot compete in cost with chemicals where freights are moderate; in regions where electric current is cheap and freights high, it may be employed to advantage. Addition of iron electrolytically may require the addition of lime to reduce the CO_2 and precipitate the hydrate.

Copper sulfate is used as an algacide. The commercial salt has the formula $CuSO_4 + 5H_2O$, and is known as "blue vitriol." Ferrous sulfate containing 1 per cent. of copper sulfate has been used at Marietta, Ohio, as a coagulant. Ten p.p.m. of copper sulfate will destroy B. coli.

Lime is used for water softening; for an alkali with ferrous sulfate and sulfate of alumina; and to neutralize acid waters. Pure lime has the formula CaO, but ordinary building lime may contain as little as 75 per cent. For water softening, the lime should not contain less than 85 per cent. of soluble CaO, and preferably more than 90 per cent. The presence of calcium carbonate, $CaCO_3$, is not disadvantageous for neutralizing acid waters. Lime should be fresh and crushed or ground into lumps smaller than 2 in. diam. It should be stored in tight bins to prevent air-slaking.

Hydrated lime is quicklime which has been slaked by steam. It is more convenient to use than quicklime, especially for dry feeding. It does not require slaking, and does not deteriorate by storage. It is usually purer than most quicklimes. Hydrated lime costs more than quicklime, largely because of its increased weight (32 per cent.) due to hydration, and consequent increased freight charges.

Soda ash, anhydrous sodium carbonate (Na_2CO_3) containing not less than 58 per cent. Na_2O or 98 per cent. of Na_2CO_3, is used to furnish alkali to waters which are deficient therein and to soften water having "permanent" or "mineral acid" hardness. It should be furnished in powdered form containing no large lumps or crystals, and not more than 0.5 per cent. of matter insoluble in distilled water. Theoretically, 1 part of sulfate of alumina requires 0.5 part of sodium carbonate to precipitate it. With soft, colored waters only about one-half of the theoretical amount should be added lest the color be redissolved. If double the theoretical amount be added to hard waters, no free CO_2 is formed. Soda ash is the best alkali to use in small plants. It is more convenient than lime and requires a shorter period of coagulation. Its cost is more than three times that of lime, for equal weights.

Caustic soda is also sometimes used for softening waters containing "permanent" or "mineral acid" hardness. It costs from 2 to 2.5 times as much as soda ash and is useful only where small quantities are required, where the presence of carbonates is undesirable or where a speedy reaction is desired. Caustic soda is sometimes added to hot boiler feed waters. It reacts very quickly, and the precipitate formed may be removed by a filter press or pressure filter, or to a great extent in a feed water heater. Caustic soda is furnished in drums. The best caustic soda contains about 76 per cent. of Na_2O.

Barium carbonate is sometimes used to remove calcium sulfate ($CaSO_4$) from water. Barium carbonate has the formula $BaCO_3$, and is practically

insoluble in water free from CO_2. In the presence of water containing $CaSO_4$, however, $BaCO_3$ reacts according to the following formula:

$$BaCO_3 + CaSO_4 = CaCO_3 + BaSO_4$$

Barium carbonate is a more expensive softening chemical than sodium carbonate, but the latter has the disadvantage of producing sodium sulfate (Na_2SO_4) as a by-product, which has a tendency to cause foaming in steam boilers unless the boilers be blown off frequently, while the insoluble $CaCO_3$ and $BaSO_4$ are precipitated during softening. Soluble barium salts are poisonous.

Bleaching powder, sometimes called chloride of lime, chlorinated lime or calcium hypochlorite, is made by saturating quicklime with chlorine. It is an active sterilizing and oxidizing agent and usually contains 30 per cent. of impurities, chiefly lime and water. It loses strength rapidly if exposed to the air. Its available ingredients have the formula $CaCl_2 + Ca(OCl)_2$. The $CaCl_2$ is inactive, but the $Ca(OCl)_2$ is decomposed by the carbonic acid in the water into hypochlorous acid (HOCl) and $CaCO_3$. The former compound is unstable and gives up its oxygen to organic matter with the production of hydrochloric acid, which latter is neutralized by the alkalinity of the water. Strength of bleaching powder is measured in terms of available chlorine. The actual oxygen liberated according to the formula is 22.5 per cent. of the available chlorine. Bleaching powder should contain not less than 35 per cent. of available chlorine. In small filter plants, bleaching powder may be mixed directly with sulfate of alumina. Dr. Lederer has suggested that waters which have been treated with too much bleaching powder may be made palatable by the addition of *sodium thiosulfate*, the equivalent of one-half of the bleaching powder being added. Because thiosulfate destroys the disinfecting power of chlorine, sufficient time should elapse before its addition; otherwise disinfection will not occur. It is essential that the chlorine be intimately mixed with the treated water in order to secure the desired effect.

Liquid chlorine may be used in place of bleaching powder. It is marketed in cylinders containing 100 lb. of chlorine which is 99.8 per cent. pure. Pressure in the cylinders varies between 50 and 100 lb. per sq. in. The application of liquid chlorine is much more facile than the application of bleaching powder; the apparatus requires much less room and the discomfort and corrosion due to dust and fumes are avoided. Theoretically, liquid chlorine is three times as strong as bleaching powder; in practice, it is difficult to use all the chlorine in bleaching powder; 1 part of liquid chlorine may do the work of 4 or 5 parts of bleaching powder; liquid chlorine costs from four to six times as much, but this cost is liable to decrease with the demand and economies in manufacture. It is rapidly taking the place of bleaching powder, even in plants as large as Torresdale, Philadelphia.

Quantities of Chemical. The quantities of *sulfate of alumina* required vary greatly with the water. Silt and colloidal coloring matter on the verge of precipitation, require but little coagulant. Freshly dissolved vegetable coloring matter and colloidal clay require much greater amounts. The turbid and slightly colored waters of South Atlantic seaboard are examples of the first class; the waters of swamps and the Mississippi river at New Orleans of the second. Contaminated waters of low turbidity and color require for the

removal of bacteria a larger dose of sulfate of alumina than necessary to coagulate the water. When waters of high color or turbidity are sufficiently decolorized or clarified, removal of bacteria is adequate. It is often economical to add chlorine to insure the safety of the water where coagulation can be accomplished by smaller doses of chemicals than would be required for the sufficient removal of bacteria. It is best to determine the required quantities of chemicals by means of a number of laboratory experiments with samples of water to which various amounts of coagulant have been added and the results noted.

Table 261. Quantities of Sulfate of Alumina (17.5 per Cent.) Required to Coagulate Waters of Various Turbidities

Turbidity, p.p m.	Basic sulfate of alumina : 17.5% Al_2O_3 required								
	Parts per 1,000,000			Grs. per gal.			Lbs. per 1,000,000 gal		
	Av.	Max.	Min.	Av.	Max	Min	Av.	Max.	Min.
Under 10	10	17	5	0.60	1.00	0.30	86	143	43
15	14	20	8	0 83	1.19	0.48	119	170	69
20	17	22	11	1.00	1.30	0.61	143	186	87
40	19	25	13	1.12	1.47	0.74	160	210	106
60	21	28	14	1.21	1.61	0.83	173	230	119
80	22	30	15	1.29	1.74	0.90	184	249	129
100	24	32	16	1.39	1.87	0.96	199	267	137
120	25	34	17	1.45	2.00	1.01	207	286	144
150	27	37	18	1.59	2.18	1.07	227	311	153
200	30	42	19	1.76	2.47	1.14	251	353	163
250	33	47	20	1.92	2.74	1.19	274	391	170
300	36	51	21	2.08	2.99	1.23	297	427	176
400	39	62	22	2.25	3.60	1.28	321	514	183
500	42	70	23	2.45	4.08	1.33	350	583	190
600	47	77	24	2.75	4.50	1.37	393	643	196
700	50	——	24	2.92	——	1.41	417	——	201
800	53	——	25	3.06	——	1.45	437	——	207
900	55	——	26	3.19	——	1.49	456	——	213
1000	56	——	26	3 28	——	1.52	469	——	217

Table 262. Average Quantities of Sulfate of Alumina (17.5 per Cent.) Required to Coagulate Waters of Various Colors

Color, p.p m.	Average sulfate of alumina required		
	Parts per 1,000,000	Grs. per gal.	Lbs. per 1,000,000 gal.
Under 10	14	0.81	116
20	19	1.13	161
40	27	1.59	227
60	33	1.93	276
80	38	2.22	317
100	42	2.45	350
120	45	2.64	377
150	50	2.90	414
200	57	3.30	471

Table 263. Average Quantities of Ferrous Sulfate and Lime Required for Various Turbidities

Turbidity of settled water	Cincinnati (1913–14)				New Orleans (1913–14)			
	Ferrous sulfate		Lime		Ferrous sulfate		Lime	
P.p.m.	P.p.m.	Lbs., mg	P.p.m.	Lbs., mg.	P.p.m.	Lbs., mg.	P.p.m.	Lbs., mg
50	30	247	14	120	...	...	...	...
100	35	289	16	138	0.3	3	56	471
150	37	310	17	145	0.9	7	65	543
200	40	333	18	151	1.2	10	70	582
250	42	353	19	160	1.5	13	74	617
300	...	...	...	...	1.9	16	77	640
400	...	...	...	...	2.7	23	82	683
500	...	...	...	...	3.6	30	85	709
600	...	...	...	...	4.3	36	88	732
700	...	...	...	...	5.0	42	90	748
800	...	...	...	...	5.8	48	91	760
900	...	...	...	...	6.5	54	92	772
1000	...	...	...	...	7.4	61	94	783
1200	...	...	...	...	8.9	74	97	808
1400	...	...	...	...	10.4	87	100	832
1600	...	...	...	...	11.8	98	102	852
1800	...	...	...	...	13.4	111	105	877
2000	...	...	...	...	14.9	124	108	898

Quantities of *ferrous sulfate* and *lime* vary greatly. Addition of lime alone, particularly to waters high in magnesia, may coagulate the water more or less perfectly, thereby reducing the dose of ferrous sulfate required. Table 263,

Table 264. Quantities of Copper Sulfate Required for Different Organisms

Organisms	Parts per mil	Lbs. per mil. gal. of water	Organisms	Parts per mil	Lbs. per mil. gal. of water
Diatomaceæ:			Cyanophyceæ:		
Asterionella	0.10	0.8	Anabæna	0.10	0.8
Fragilaria	0.25	2.1	Clathrocystis	0.10	0.8
Melosira	0.30	2.5	Cœlosphærium	0.30	2.5
Synedra	1.00	8.3	Oscillaria	0.20	1.7
Navicula	0.07	0.6	Microcystis	0.20	1.7
Chlorophyceæ:			Aphanizomenon	0.15	1.2
Cladophora	1.00	8.3	Protozoa:		
Conferva	1.00	8.3	Euglena	0.50	4.2
Hydrodictyon	0.10	0.8	Uroglena	0.05	0.4
Scenedesmus	0.30	2.5	Peridinium	2.00	16.6
Spirogyra	0.20	1.7	Glenodinium	0.50	4.2
Ulothrix	0.20	1.7	Chlamydomonas	0.50	4.2
Volvox	0.25	2.1	Cryptomonas	0.50	4.2
Zygnema	0.70	5.8	Mallomonas	0.50	4.2
Microspora	0.40	3.3	Dinobryon	0.30	2.5
Draparnaldia	0.30	2.5	Synura	0.10	0.8
Raphidium	0.30	2.5	Schizomycetes:		
Cœlastrum	0.30	2.5	Beggiatoa	5.00	41.5
			Cladothrix	0.20	1.7
			Crenothrix	0.30	2.5
			Leptomitus	0.40	3.3

based on experience at Cincinnati, and New Orleans (1913–1914), gives the average quantities of ferrous sulfate and lime required. For quantities of chemicals required to soften water, see pp. 719 and 721.

Quantities of *copper sulfate* required for different organisms, according to Kellermann (Bul. 76 and Bul. 100, 1906, Bur. of Plant Industry, U. S. Dept. of Agri.; also Proc. 8th Internat. Cong. App. Chem., **26**, 1912, 241) are given in Table 264.

Quantities in Table 264 should be increased or decreased 2.5 per cent. for each degree C. below and above 15°. If organic matter and alkalinity be high, or if carbonic acid be low, the quantity should be increased further. Kellermann recommends that the limits given in Table 265 should be set to prevent killing certain fish.

Table 265. Safe Limits for Treating Water with $CuSo_4$ to Prevent Killing Fish

Name of fish	Parts per million	Pounds per million gallons (approximate)
Trout	0.14	1.2
Carp	0.30	2.5
Suckers	0.30	2.5
Catfish	0.40	3.5
Pickerel	0.40	3.5
Goldfish	0.50	4.0
Perch	0.75	6.0
Sunfish	1.20	10.0
Black bass	2.10	17.0

Computing Volume of Water to be Treated. If the body of water treated is so deep that the lower strata are stagnant, the volume of water treated should include only that above and within the transition zone. (See Limnology, p. 676.) Approximate volume of water to be treated in a reservoir or pond in million gallons = $\frac{1}{3}$ × area in acres × average depth in ft.

Devices for Adding Chemicals. It is essential that the coagulant, whether added in solution or in the dry form, be accurately proportioned to the quantity of water treated. The flow of the treated water being known, the dose of chemical may be varied by hand from time to time, or automatically by numerous devices.

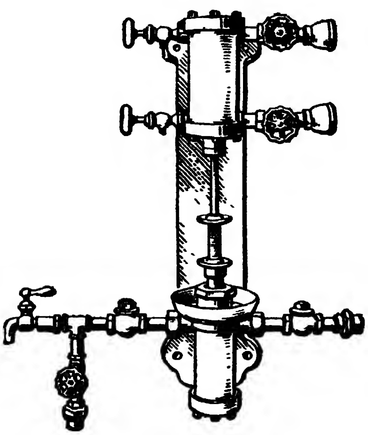


FIG. 353.

Solutions. Where a reciprocating pump is used to lift the water, a small coagulant pump may be attached to the valve rod, or a Hodkinson, variable-stroke, synchronous, chemical feed pump may be used. Fig. 353 shows this pump, which is a hydraulic pump actuated by the hydraulic pressure generated alternately from the two ends of the reciprocating pump chamber. The stroke of this pump can be

varied. Where the water to be coagulated is discharged through a pipe under pressure, the coagulant feed pump may be operated by a turbine placed within the pipe. This device is not reliable for greatly varying flows. Where coagulant is added by gravity, the principle of an orifice discharging from a tank in which the solution is maintained at a constant level may be employed. Fig. 354 shows a cross-section through solution tank and orifice tank. The chemicals to be dissolved are placed in the solution box and dissolved by water from the tap (A). The solution tank is filled to the flow line and mixed by compressed air from pipe (B). (A mechanical stirrer may be employed.) The mixed solution is fed into the

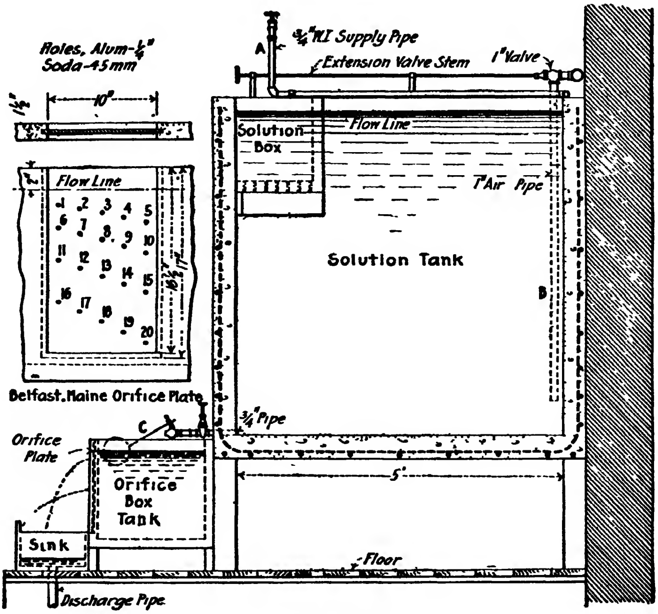


FIG. 354.

orifice tank through the ball-cock (C), which maintains the solution at a constant level. In the front of the tank is a glass plate drilled with 20 counter-sunk holes closed with rubber stoppers. One or more of these orifices can be used to discharge the solution into the porcelain sink which communicates with the pipe discharging into the water to be treated. Instead of the orifice plate, one of the various feed valves or adjustable slots may be used. Small orifices or valves are liable to clog. There should be a low-water alarm consisting of gage contact points and electric bell to detect interruption in the discharge, or where the raw water is delivered at a constant rate by motor-driven centrifugal pumps, the device shown in Fig. 355 may be used. Here the solution tank is similar to that used in the gravity-type feeder. The solution is discharged through an orifice box by a motor-driven pump which is started and stopped by the same switch which controls the raw-water pump. The solution overflows from the feed tank whenever the raw-water pump is op-

erated. Attached to the feed tank is an outlet of some sort. The one shown in Fig. 355 consists of a flexible tube connected with a calibrated glass tube which can be raised and lowered on a graduated scale. This tube discharges into a pipe connected with the raw-water main. Whenever the raw-water pump stops, the coagulant pump stops also, the feed tank quickly drains and the flow stops; and *vice versa* when the raw-water pump starts.

The first automatic chemical feed devices based on the Venturi meter were installed at Columbus, Ohio, in 1908, after designs by Gregory, Jackson and Connet (U. S. Patent No. 868776, Oct. 22, 1907). In this device the floats actuated by the meter tube operated a cam, which in turn controlled the

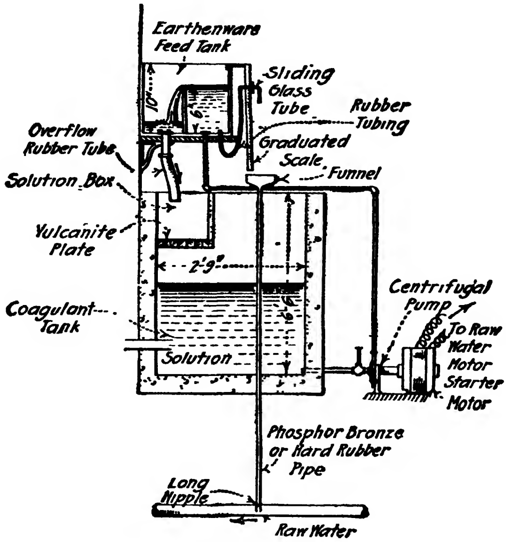
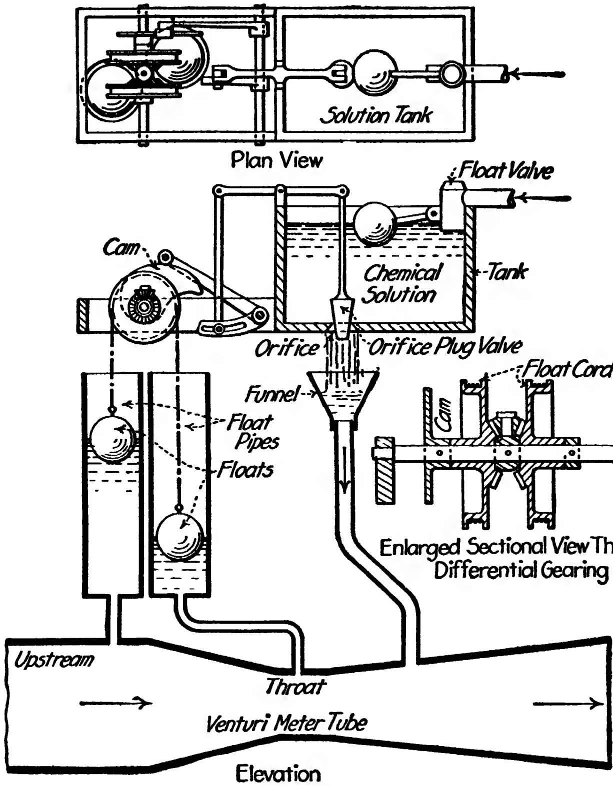


FIG. 355.

orifice discharging from a constant-head tank. Recently at Baltimore the cam has been used to start and stop a small electric motor, which in turn controls the raising and lowering of the plug in a circular orifice discharging from a constant-head tank. At Cairo, Egypt, the plug is raised and lowered directly by means of the cam. Fig. 356 illustrates this type of apparatus. It is made by the Builders Iron Foundry.

A similar device, of New Orleans type, is shown in Fig. 357. There are three float tubes—A and B connected respectively with the upstream end and throat of a Venturi meter through which the raw water passes; C connected with a feed tank of the gravity ball-cock type. The three floats, F_1, connect with a lever, L, supported on the fulcrum P. One end of the lever is connected with an automatic valve, V, which admits coagulant solution into the tank C. A pipe connects C with the feed tank T. This system of levers and valve maintains a head over the orifice in the feed tank equivalent to the difference in elevation between the water-levels in float tubes A and B, thus discharging the coagulant solution in direct proportion to the flow.



DIAGRAMMATIC VIEW OF VENTURI CHEMICAL FEED DEVICE

FIG. 356.

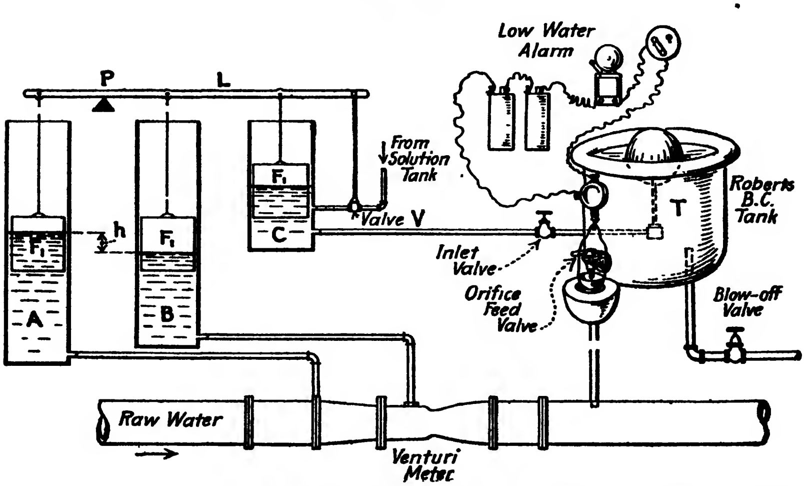


FIG. 357.—Venturi type coagulant feed controller. (Stein.) Orifice feed valve must be at a hight above the main greater than the head in the main at the point of connection.

Besides the above, proportional piston or rotary meters are sometimes used.

Tanks holding *acid coagulant solutions* should be built of concrete or enamel-lined iron and the piping should be of rubber, glass, earthenware or acid resistant metal. Phosphor-bronze withstands sulfate of alumina best of all alloys. Alkaline solutions may be stored in iron or concrete tanks and fed through iron piping. To prevent clogging, pipes handling lime or milk of lime should be of ample diameter and provided with numerous clean-outs. For feeding milk of lime, the Sutro weir and chemical regulator used with the Booth water softener (Trans. Am. Inst. Chem. Engrs., Vol. 7, 1914) are well adapted. The Sutro weir, Fig. 358, is designed so that head above crest is directly proportional to discharge. A float is placed in the weir chamber and connected by bell-crank levers and rods with the cut-off plate of the chemical regulator, Fig. 359. This latter is a proportional weir in which the lime suspension is kept in constant agitation and therefore homogeneous. A constant head is maintained over the discharge of weir, and movements of the cut-off plate cause the discharge of varying amounts of solution into the raw water, unused portions being returned to the solution tank. Lime may be fed by a Venturi controller like one used at New Orleans. Where chemicals are added to small filters, a differential feeding apparatus may be used. In the Roberts apparatus, Fig. 360, the feeding ports are prevented from clogging by spiral brass rods which are moved up and down through the ports every time the adjusting handle attached to the pointer shown on the graduated dial is moved.

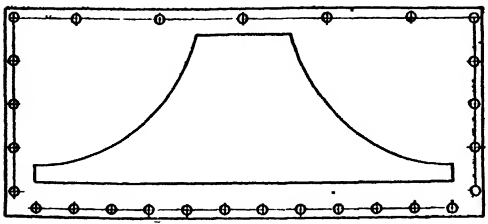


Fig. 358.

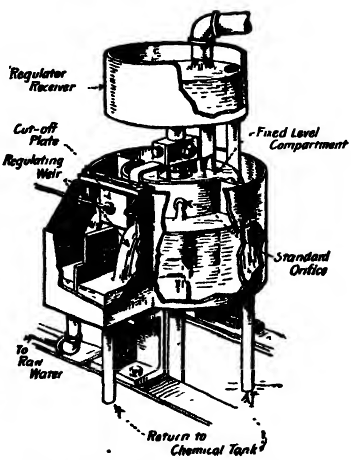


Fig. 359.—Chemical regulator. (Booth.)

Bleaching powder is difficult to apply to water. It is first mixed with a smaller quantity of water—not less than 0.5 gal. water per lb. of bleach—and this "paste" thoroughly mixed and diluted to make a solution containing not over 2 per cent. of bleach. Considerable sludge is produced, and the chlorine in this should be recovered by agitating it with water and using the water after the subsidence of the sludge to make up the next batch of solution. Tanks for bleach solution are best made of reinforced concrete or pure wrought iron. Piping may be of pure black wrought iron, lead, stone-ware or glass; fittings should be of stone-ware or acid-proof bronze. Avoid wood, copper, ordinary brass

and steel. The solution should be kept constantly agitated until applied to the water.

Liquid chlorine is added directly from cylinders by means of a regulating apparatus. Fig. 361 shows a two-cylinder apparatus arranged for hand feeding. It consists of a manifold to which two chlorine cylinders may be

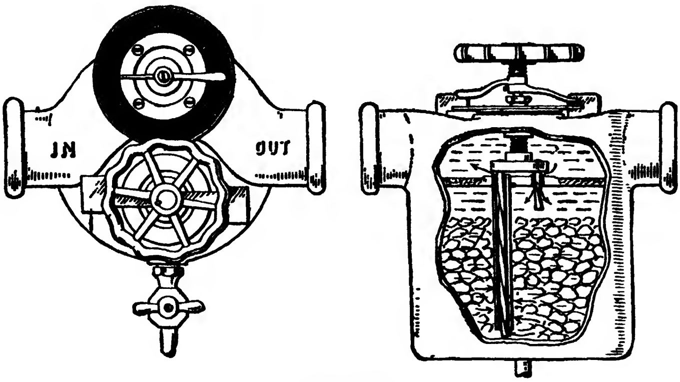


FIG. 360.

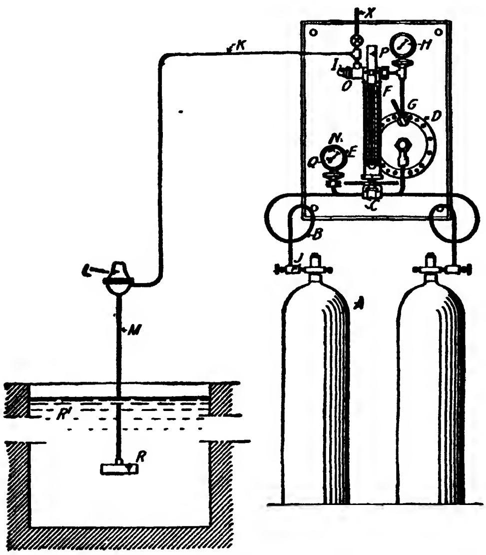


FIG. 361.
(Wallace & Tiernan Co.)

connected. Chlorine gas in a cylinder (*A*) is fed through an orifice (*O*), measured in a float meter (*F*) and piped through a tube (*K*), a check valve (*L*) and a silver tube (*M*) to the diffuser (*R*); this latter consists of a carborundum sponge of fine porosity. The gas pressure forces the gas through the pores of the carborundum where the minute bubbles become saturated with moisture. When the chlorine bubbles reach the water they are dissolved im-

mediately, not only because of their fineness, but also because they are already saturated with moisture. An automatic valve (*C*) automatically cuts in a second cylinder as soon as the first cylinder becomes exhausted.

Fig. 362 shows an apparatus designed to maintain a proportional flow of chlorine and water. This device makes use of the Venturi meter tube under the patent of Gregory, Jackson and Connet. The feeding apparatus can be connected to a Venturi meter recorder, as shown in Fig. 362, or to independent mercury columns actuated by the pressures in up-stream and throat chambers of a Venturi tube. The flow of chlorine is accurately proportioned to the flow of water by means of an electrical mechanism, the contact points of which are connected with the meter mechanism. The electrical mechanism controls the valve which varies the flow of chlorine. The contact points can also be controlled by the variable head flowing over a weir or the differential head on a submerged orifice. The power for operating the electrical device can be obtained from a lighting circuit or from storage batteries.

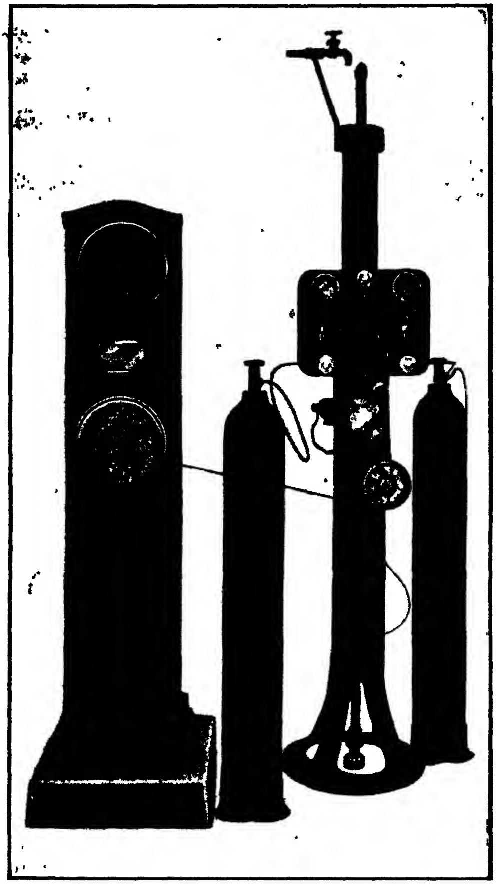


FIG. 362.
(Electro Bleaching Gas Co)

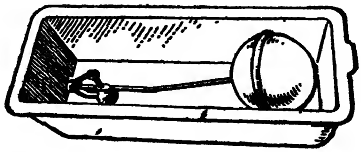


FIG. 363.—Pump suction box.
(New York, Continental, Jewell.)

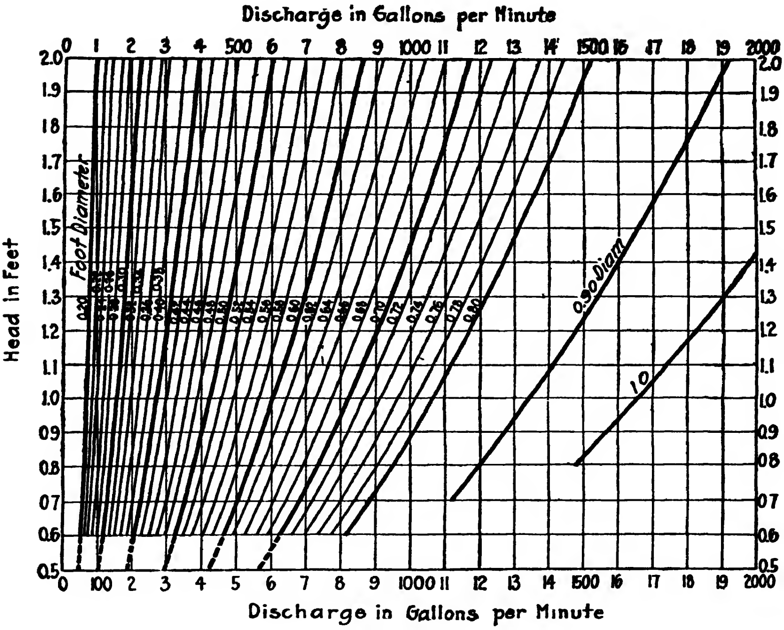


FIG. 364.

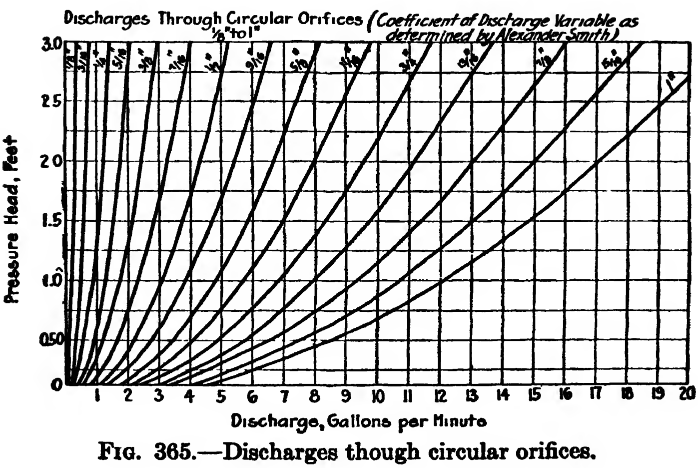


FIG. 365.—Discharges though circular orifices.

Sodium hypochlorite, prepared by electrolyzing a solution of sodium chloride, has been used in place of a solution of bleaching powder. The brine to be electrolyzed is led through a special electric cell where a complicated reaction takes place, which may be partly represented by the following formula:

$$NaCl + H_2O + \text{current} = NaOCl + H_2$$

First, the sodium is liberated at one pole, the chlorine at the other. Sodium then decomposes the water, forming sodium hydroxide and hydrogen. Chlorine combines with sodium hydroxide, forming sodium hypochlorite and hydrogen. Hydrogen escapes. The solution discharged from the cell may be applied to water like any other chemical solution. One hp. per 24 hrs. can produce from 10 to 12 lbs. of chlorine, as NaOCl, requiring the use of 25 to 30 lbs. of salt.

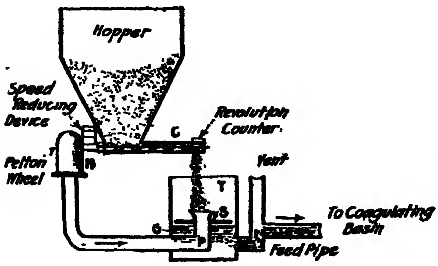


Fig. 366.

Where *chemical solutions* are discharged *into pump suctions*, a pump suction box, Fig. 363, may be used.

Discharges of Orifices and Weirs. Table* 266 gives discharges of a V-notch weir under small heads. Discharges of weirs with rectangular notches are given on page 598, and of circular orifices in Figs. 364 and 365. Rates of flow of chemical solutions of different strengths which are required for the addition of ten parts of chemical per million for different rates of filtration are given in Table 267.

Table 266. Discharges over a 90° Triangular Notch Weir

$Q = 2.64h^{\frac{5}{2}}$(Q in cfs.; H in ft.)

h, inches	Gallons per min.	Liters per min.	h, inches	Gallons per min.	Liters per min.
1/4	0.076	0.29	3 1/4	45.1	171
3/8	0.42	1.6	3/8	49.7	188
1/2	0.74	2.8	1/2	54.4	206
5/8	1.16	4.4	5/8	59.2	224
3/4	1.71	6.5	3/4	64.6	245
7/8	2.38	9.0	7/8	70.1	266
1	3.19	12.1	4	75.9	288
1/8	4.17	15.8	1/8	82.1	311
1/4	5.25	19.9	1/4	88.3	335
3/8	6.55	24.8	3/8	95.1	360
1/2	7.99	30.3	1/2	102.0	387
5/8	9.60	36.4	5/8	109.2	414
3/4	11.4	43.0	3/4	116.7	442
7/8	13.4	51.0	7/8	124.6	472
2	15.7	60.0	5	132.7	503
1/8	18.0	68.0	1/8	141.1	535
1/4	20.6	78.0	1/4	150.0	569
3/8	23.5	89.0	3/8	159.0	603
1/2	26.5	100.0	1/2	168.5	639
5/8	30.1	114.0	5/8	178.3	676
3/4	33.4	127.0	3/4	188.3	714
7/8	37.0	140.0	7/8	198.8	753
3	41.0	155.0	6	209.5	794

* See also p. 600.

Table 267. Rates of Flow of Chemical Solutions of Different Strengths for Different Rates of Filtration

U. S. gallons		Gallons of chemical solution per min. to add 10 parts of chemical per million						
24 hours	Min.	1%	2%	3%	4%	5%	6%	10%
250,000	174	0.174	0.087	0.058	0.044	0.035	0.029	0.017
500,000	347	0.347	0.174	0.116	0.087	0.069	0.058	0.035
1,000,000	694	0.694	0.347	0.231	0.174	0.139	0.116	0.069
1,500,000	1,042	1.042	0.521	0.347	0.260	0.208	0.174	0.104
2,000,000	1,389	1.389	0.694	0.463	0.347	0.278	0.232	0.139
2,500,000	1,736	1.736	0.868	0.579	0.434	0.347	0.289	0.174
3,000,000	2,083	2.083	1.042	0.694	0.521	0.416	0.347	0.208
4,000,000	2,778	2.778	1.389	0.926	0.694	0.556	0.463	0.278
5,000,000	3,472	3.472	1.736	1.157	0.868	0.694	0.579	0.347
7,500,000	5,208	5.208	2.604	1.736	1.302	1.042	0.868	0.521
10,000,000	6,944	6.944	3.472	2.315	1.736	1.389	1.157	0.694
20,000,000	13,889	13.889	6.944	4.630	3.467	2.778	2.315	1.389

Dry Chemicals. In many plants dry feeding of chemicals is practicable (Hazen, E. R., July 19, 1913; and Monfort, Jour. A. W. W. A., 2, 1915, 200). Sulfate of alumina, quicklime, hydrated lime and ferrous sulfate may be fed dry if the chemicals are ground and means are provided to crush any lumps which may form during storage. Ferrous sulfate particularly is apt to cake when stored in bins; it should therefore be freshly crushed and fed quickly. Dry feeding avoids solution tanks, orifices and pipes and economizes space. It permits a ready control of applied chemicals. Fig. 366 illustrates a dry feeding

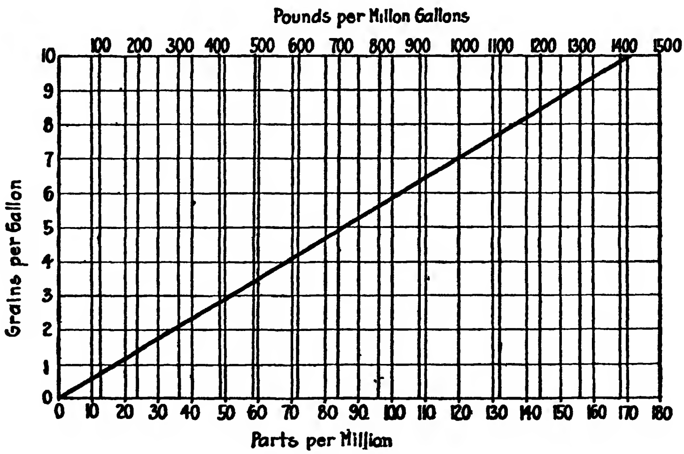


FIG. 367.—Conversion diagram. Parts per million, pounds per million gallons, grains per gallon.

apparatus. It consists of a hopper discharging into the helical conveyor *C*, driven by an electric motor or Pelton wheel, *M*. Suitable speed-regulating devices are used to govern the revolutions of the conveyor and consequently the discharge of coagulant. The conveyor is calibrated and the quantity of chemical is measured by the number of revolutions, checked by weighing the chemical before placing in the hopper. The chemical discharges into the

upturned end of the raw-water pipe, *P*, in the chamber, *T*. A grating, *G*, prevents the coarser lumps from settling to the bottom of the chamber before they are dissolved. Spray pipes, *S*, are sometimes used to facilitate the solution of the chemicals. The treated water passes from the chamber into the coagulating basin.

Where large quantities of lime or soda are used for water softening, automatic weighing devices (discharging intermittently) may be used in place of continuous dry feeders. The plant at Columbus, Ohio (Ann. Rep., Div. of Water, City of Columbus, 1914), is designed to add lime in the form of milk of lime containing from 3 to 4 per cent. of CaO. Compressed air is used for stirring this suspension. Lime is received and stored in bulk, and is fed into the three lime slaking tanks from three small hoppers. Each of these hoppers discharges into an automatic weighing device, electrically controlled, which weighs out the desired quantity of lime at regular intervals into the slaking tanks, whence the lime suspension passes into the solution tank and thence, with the added water, into the stream of raw water in the mixing tanks. The Richardson automatic timing device, which operates the automatic scales, consists of a clockwork which uniformly rotates a circular disk with contact points on its circumference. Every time a contact is made, a charge of lime is dumped by the automatic scales. Different disks are used to give any desired interval between contacts. The equipment is designed so that if advisable at any time the slaked lime may be added directly to the water without first passing through the solution tanks. This is practically the method used at St. Louis, where weighted amounts of lime are added at definite intervals of time.

Applying copper sulfate is usually effected by placing in a bag, perforated bucket or wire basket, attached to a rope and dragged back and forth at the stern of a boat. One or more bags may be used at the same time. Speed of boat and rate of addition of chemical to the bucket or bag may be so regulated that not over 100 lb. will be dissolved in an hour. The boat should move fast enough to avoid too great concentration near the bag, that the killing of fish may be avoided. It is better to apply the chemical when the wind is blowing.

Period of coagulation for any given water is best determined by experiment, and varies from 0.25 to 6 hr. for sulfate of alumina and the like and from 4 to 12 hr. for softening chemicals. The period of coagulation for water purified with modified, slow filters, *e.g.*, Washington, should be long enough to effect the practically complete removal of all the coagulum in the basin. Usually 24 hr. is more than ample. Waters applied to mechanical filters should not be given so long a period of coagulation that no coagulum will be left to form suitable films in the filter. An increase in the period of coagulation often reduces the quantity of required coagulant. Quantities of sulfate of alumina required for the Mississippi river water at New Orleans, after subsidence, were as follows:

Period of coagulation	Sulfate of alumina, p.p.m.
6 hrs.	75
12 hrs.	70
24 hrs.	70

Table 268. Costs of Chemicals Used as Coagulant per 1,000,000 Gals. of Treated Water

P.p.m.	Grs. per gal.	Price of chemical per 100 lbs. $0.75	$0.80	$0.85	$0.90	$0.95	$1.00	$1.05	$1.10	$1.15	$1.20	$1.25	$1.30	$1.35	$1.40	$1.45	$1.50	$1.55	$1.60	$1.70	$1.75	Pounds per million gals.
1	0.06	0.062	0.066	0.070	0.075	0.079	0.083	0.087	0.091	0.095	0.100	0.104	0.108	0.112	0.116	0.120	0.124	0.129	0.133	0.141	0.145	8.3
5	0.29	0.313	0.334	0.355	0.375	0.396	0.417	0.438	0.459	0.480	0.501	0.522	0.542	0.563	0.584	0.605	0.626	0.647	0.668	0.709	0.730	42.0
10	0.59	0.626	0.668	0.709	0.751	0.793	0.834	0.876	0.918	0.960	1.00	1.04	1.08	1.13	1.17	1.21	1.25	1.29	1.34	1.42	1.46	83.0
15	0.88	0.939	1.00	1.06	1.13	1.19	1.25	1.31	1.37	1.44	1.50	1.56	1.63	1.69	1.75	1.81	1.88	1.94	2.00	2.13	2.19	125.0
20	1.17	1.25	1.34	1.42	1.50	1.59	1.67	1.75	1.84	1.92	2.00	2.09	2.17	2.25	2.34	2.42	2.50	2.59	2.67	2.84	2.92	167.0
25	1.46	1.56	1.67	1.77	1.88	1.98	2.09	2.19	2.29	2.40	2.50	2.61	2.71	2.82	2.92	3.02	3.13	3.23	3.34	3.55	3.65	209.0
30	1.75	1.88	2.00	2.13	2.25	2.38	2.50	2.63	2.75	2.88	3.00	3.13	3.25	3.38	3.50	3.63	3.75	3.88	4.00	4.26	4.38	250.0
35	2.05	2.19	2.34	2.48	2.63	2.77	2.92	3.07	3.21	3.36	3.50	3.65	3.80	3.94	4.09	4.23	4.38	4.53	4.67	4.96	5.11	292.0
40	2.34	2.50	2.67	2.84	3.00	3.17	3.34	3.50	3.67	3.84	4.01	4.17	4.34	4.51	4.67	4.84	5.01	5.17	5.34	5.67	5.84	334.0
45	2.63	2.62	3.00	3.10	3.38	3.57	3.75	3.94	4.13	4.32	4.51	4.69	4.88	5.07	5.26	5.44	5.63	5.82	6.00	6.38	6.57	375.0
50	2.92	3.13	3.34	3.55	3.75	3.97	4.17	4.38	4.59	4.80	5.01	5.22	5.43	5.63	5.84	6.05	6.26	6.47	6.68	7.09	7.30	417.0
55	3.21	3.44	3.67	3.90	4.13	4.36	4.59	4.82	5.05	5.28	5.51	5.74	5.97	6.20	6.42	6.65	6.88	7.11	7.34	7.80	8.03	459.0
60	3.50	3.75	4.01	4.26	4.51	4.76	5.01	5.26	5.51	5.76	6.01	6.26	6.51	6.76	7.01	7.26	7.51	7.76	8.01	8.51	8.76	501.0
65	3.80	4.07	4.34	4.61	4.88	5.15	5.42	5.69	5.97	6.24	6.51	6.78	7.05	7.32	7.59	7.86	8.14	8.41	8.68	9.22	9.49	542.0
70	4.09	4.38	4.67	4.96	5.26	5.55	5.84	6.13	6.42	6.72	7.01	7.30	7.59	7.89	8.18	8.47	8.76	9.05	9.35	9.93	10.22	584.0
75	4.38	4.69	5.01	5.32	5.63	5.94	6.26	6.57	6.88	7.20	7.51	7.82	8.14	8.45	8.76	9.07	9.39	9.70	10.01	10.64	10.95	626.0
80	4.67	5.01	5.34	5.67	6.01	6.34	6.68	7.01	7.34	7.68	8.01	8.34	8.67	9.01	9.35	9.68	10.01	10.35	10.68	11.35	11.68	667.0
85	4.97	5.32	5.67	6.03	6.38	6.74	7.09	7.45	7.80	8.16	8.51	8.87	9.22	9.57	9.93	10.28	10.64	10.99	11.35	12.06	12.11	709.0
90	5.26	5.63	6.01	6.38	6.76	7.13	7.51	7.89	8.26	8.64	9.01	9.39	9.76	10.14	10.51	10.89	11.26	11.64	12.02	12.77	13.14	751.0
95	5.55	5.95	6.34	6.74	7.13	7.53	7.93	8.32	8.72	9.12	9.51	9.91	10.30	10.70	11.10	11.49	11.89	12.29	12.68	13.47	13.87	793.0
100	5.84	6.26	6.68	7.09	7.51	7.93	8.34	8.76	9.18	9.60	10.01	10.43	10.85	11.26	11.68	12.10	12.52	12.93	13.35	14.18	14.60	834.0
110	6.43	6.88	7.34	7.80	8.26	8.72	9.18	9.64	10.10	10.56	11.01	11.47	11.93	12.39	12.85	13.31	13.77	14.23	14.69	15.60	16.06	918.0
120	7.01	7.51	8.01	8.51	9.01	9.51	10.01	10.51	11.01	11.51	12.02	12.52	13.02	13.52	14.02	14.52	15.02	15.52	16.02	17.02	17.52	1001.0
130	7.59	8.14	8.68	9.22	9.76	10.30	10.85	11.39	11.93	12.47	13.02	13.56	14.10	14.64	15.19	15.73	16.27	16.81	17.36	18.44	18.98	1085.0
140	8.18	8.76	9.35	9.93	10.51	11.10	11.68	12.27	12.85	13.43	14.02	14.60	15.19	15.77	16.35	16.94	17.52	18.11	18.69	19.86	20.44	1168.0
150	8.76	9.39	10.01	10.64	11.26	11.89	12.52	13.14	13.77	14.39	15.02	15.64	16.27	16.90	17.52	18.15	18.77	19.40	20.03	21.28	21.90	1251.0
200	11.68	12.52	13.35	14.18	15.02	15.85	16.69	17.52	18.36	19.19	20.03	20.86	21.69	22.53	23.36	24.20	25.03	25.87	26.70	28.37	29.20	1669.0

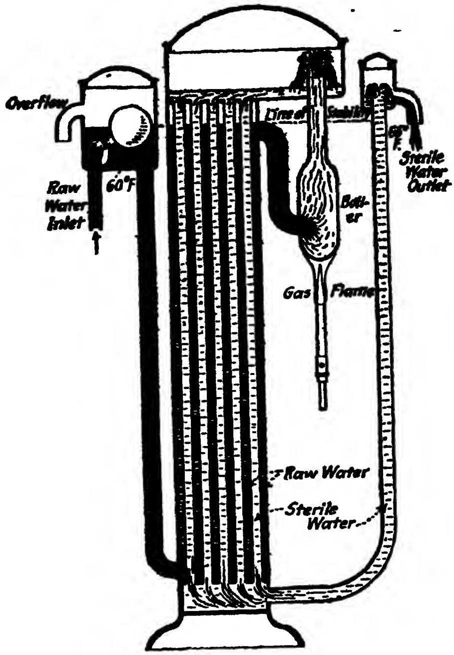


FIG. 368.

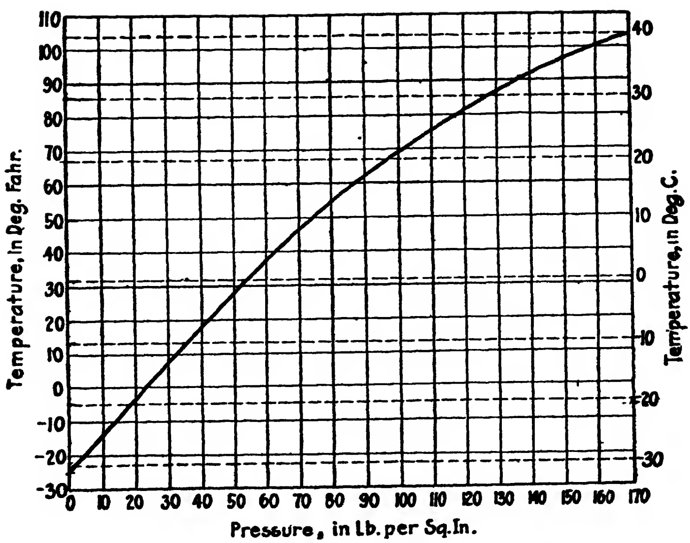


FIG. 369.—Curve showing pressure of chlorine gas at various temperatures.

DISINFECTION

Disinfection is practised to destroy disease germs. Total destruction of all bacteria by *sterilization* is not usually necessary. Disinfection may be accomplished by heat, light, application of chemicals, including ozone, and by ultra-violet rays.

Heat is an efficient disinfecting agent, but too costly where large volumes have to be treated; boiled water has a flat, insipid taste, caused by the loss of dissolved gases. To sterilize water completely, it should be boiled ½ hr.; disinfection may be accomplished by heating to 60° C. for 15 min. or by boiling for a shorter time. Sterilization may be accomplished also in sterilizers designed on the counter-current principle. In the Forbes sterilizer, illustrated schematically in Fig. 368, the water is slowly heated, boiled a few seconds and then cooled again. The hot water leaving the sterilizer is cooled by the raw water entering. The flow of water is induced by the expulsion of part of the water during the act of boiling; consequently there is no flow through the apparatus when the heat is shut off. Water must be heated to the boiling point to pass through the apparatus. The original dissolved gases and taste of the water are retained and the temperature raised less than 7° C. (13° F.). Table 269 gives the capacities, sizes of connections, and steam consumption of steam-operated Forbes sterilizers. In other types, the source of heat may be gas, electric current, alcohol, etc.

Table 269. Data for Type "S" Forbes Sterilizer
(Steam Operated)

Sterilizer	#1	#2	#3	#4	#5	#6	#7	#8
Gallons per hr.	10	25	50	75	100	150	250	500
Pipe sizes, raw water	⅜	⅜	⅜	½	¾	¾	1	1
" " steam	⅜	⅜	⅜	½	¾	¾	1	1
" " waste	1½	1½	1½	1½	2	2	2½	2½
" " sterile water	¾	¾	¾	1	1½	1½	2½	2½
Pounds steam per hr. at 5 lbs. pressure	2½	6	12	18	24	36	60	120

Temperature of raw water 70° F. Temperature of sterile water 82° F.

Oxidizing Agents. Chlorine, bleaching powder (calcium hypochlorite) and sodium hypochlorite are the most efficient and economical chemical disinfectants. Their efficiency is roughly proportional to their chlorine content.

Liquid chlorine is the most convenient disinfectant. Cl is a greenish-yellow gas, easily compressed to a liquid. Under a pressure of 6 atmospheres, it liquefies at 0° C. The use of liquid chlorine as a sterilizing agent was introduced by Maj. C. R. Darnall in 1910. The cost of disinfecting water with liquid chlorine varies between 9 cts. and 80 cts. per m.g. Ordinarily from 0.2 to 0.5 p.p.m. of chlorine is sufficient to disinfect a water, although some waters require more. Lederer and Bachmann (E. R., 67, 1913, 317) state that 0.6 p.p.m. is smallest quantity of Cl which can be detected by the ordinary observer.

Table 270. Weights of Chlorine

Datum	Gaseous chlorine*	Liquid chlorine
Specific gravity	2.491 (Air = 1)	1.44 (Water = 1)
Weight of 1 liter	3.167 grm.	1440 grm.
Weight of 1 cu. ft.	0.198 lb.	89.752 lbs.
Weight of 1 gal	0.026 lb.	11.999 lbs.

One volume of liquid chlorine is equivalent to 444.4 volumes of chlorine gas.
* Atmospheric pressure, sea level 32°F.

Table 271. Solubility of Chlorine

Temperature		Solubility ratio by volume	Pounds of chlorine soluble in 1,000,000 gals. of water
C.°	F.°		
0	32	1 5	39,000
10	50	3.0	78,000
30	88	1 8	48,600

Bleaching powder and *sodium hypochlorite* have similar properties. Former is cheapest disinfectant known; the latter is not in general use. They have the same effect. Ordinarily less than 1.5 p.p.m. of bleaching powder is all that is required to sterilize water, although as high as 4 p.p.m. may be used with certain waters without producing an objectionable taste, which is the danger with the chlorine treatment. Ten pounds of bleaching powder per m.g. is a common dose for unfiltered water, while filtered waters may require only half as much. Objectionable taste due to the compounds which chlorine forms with organic matter may be avoided: (1) by the proper proportioning of the dose; (2) by storage and aeration after treatment, which gives an opportunity for the decomposition of the objectionable chlorine compounds; (3) by filtration; (4) by the addition of sodium thiosulfate.

"*Ferrochlor*," invented by Duyk, is a combination of ferric chloride and bleaching powder. It is a combined coagulant and disinfectant. Montsouris Laboratory (Paris) showed that the addition of 20 p.p.m. of $FeCl_3$ and 3 p.p.m. of calcium hypochlorite practically sterilized Seine river water containing 1869 bacteria per cc. before treatment and filtration. While efficient, the chemical is costly. It possesses no advantages over sulfate of alumina and bleaching powder.

Light. Sunlight exerts a powerful germicidal action on bacteria in the surface waters of reservoirs. This action, even in clear waters, is of no importance at depths of more than a very few feet, unless the water be in active circulation and the action of light be exerted on successive surface layers. Light has practically no effect in turbid or highly colored waters.

Ozone is applied in the form of ozonized air, made by the silent discharge of electricity in a space through which a current of air passes. The process is ideal because sterilization is effected without the addition of any foreign matter. The largest installations are in Petrograd and at St. Maur waterworks, Paris. Devices for producing ozone are expensive and the cost of treatment is high. The process is inefficient when the water is turbid or colored and the treatment of a well purified filter effluent is seldom necessary. As a disinfectant ozone is far inferior in efficiency and economy to chlorine.

Ultra-Violet Rays. Ultra-violet rays from a mercury vapor, quartz lamp, will sterilize water completely and rapidly, provided it be exposed to the rays in thin layers, contains no interfering turbidity or color and is not supersaturated with gases. Ultra-violet rays not only destroy bacteria in the vegetative stage, but spores as well. The lamps employed are so constructed that nearly all the light produced will enter the water. Usually a number of lamps are inserted in the sides of a flume or in a properly baffled casing so as to give the water a number of exposures between inlet and outlet. See Fig. 370.

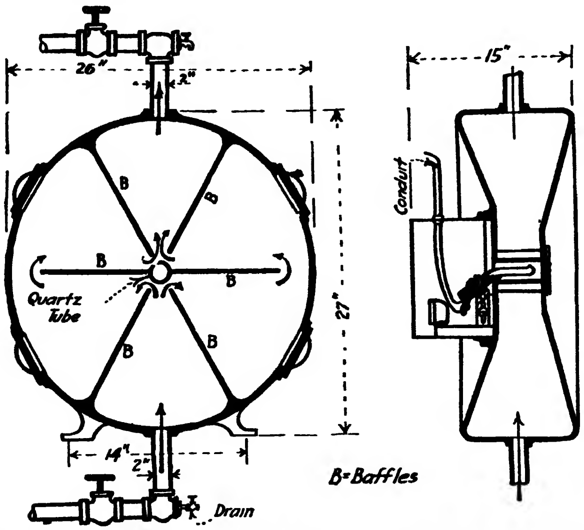


FIG. 370.—Apparatus for exposing water to ultra-violet rays.
(von Recklinghausen)

Water may be overdosed with ultra-violet rays with impunity. The method has been used in Marseilles, France, since 1910, and has been employed in the United States especially for sterilizing spring and bottled water. Ultra-violet rays cannot compete in cost with chlorine. According to von Recklinghausen, Jl. Am. W. W. Ass'n (1914), p. 583, water in large plants would require a current consumption of from 50 to 125 kw.-hr. per m.g. Ultra-violet rays and ozone are ideal for the treatment of small volumes of clear water, where the cost of operation is not an important consideration.

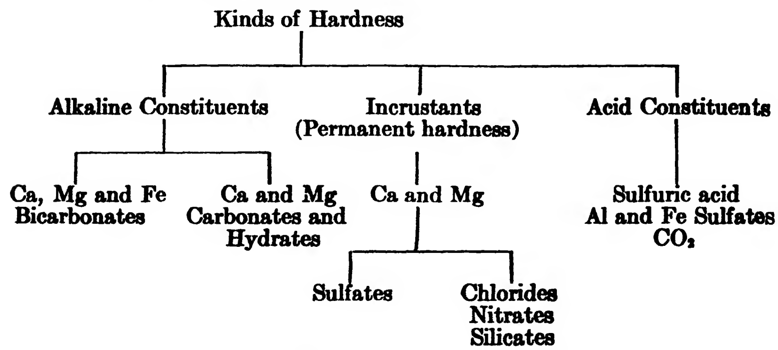
CHAPTER XXXV

WATER SOFTENING

Waters which have a high soap-consuming power or produce scale or deposits when used in steam boilers require softening.

Hard waters, so named because of their action on the skin, are those which precipitate soap. They form scale or deposits when used in steam boilers. On account of the formation of insoluble compounds with dyes and soaps, they are unsuitable for washing, bleaching, certain kinds of dyeing, soap making, steam making, tanning, wool-scouring and certain kinds of paper-making. Hard water containing calcium sulfate (selenitic water) is considered best for brewing light-colored malt beverages.

Hardness is due to the presence of the bicarbonates, carbonates, sulfates, chlorides and nitrates of calcium and magnesium, also to acid constituents. When a hard water is boiled, it loses its bicarbonate, or *temporary*, hardness, leaving the "mineral acid," or *permanent*, hardness as well as some hardness due to carbonates which cannot be so removed. *Total hardness* is the sum of temporary and permanent hardness. Waters containing free CO_2 have an apparent temporary hardness. Hardness of water polluted with mine waters is due partly to free mineral acids.



Hardness may be *expressed* in various ways, preferably as parts of $CaCO_3$ per million. Clark, the discoverer of the softening process in 1841 (Repertory of Patent Inventions, 1841) established degrees of hardness as grains of $CaCO_3$ per Imp. gal. German degrees designate parts of CaO in 100,000 and French degrees parts of $CaCO_3$ in 100,000 of water. The A. P. H. A. Committee recommend parts of $CaCO_3$ per million, Table 272. What would be called hard water in New England (*e.g.*, Reading: 120 p.p.m.), would be called soft in the limestone regions of the West. General objections to hard waters are difficulty and disadvantage of their use with soap; difficulty, cost and danger attendant upon their use for steam making.

Table 272. Equivalents of Methods of Expressing Hardness

Standard	Clark degrees	German degrees	French degrees	Parts per million	Grains per U.S. gal.
Clark degrees	1.0	0.80	1.43	14.3	0.83
German degrees	1.25	1.0	1.78	17.8	1.04
French degrees	0.7	0.56	1.0	10.0	0.583
Parts per million (milligrams per liter)	0.07	0.056	0.1	1.0	0.058
Grains per U.S. gal	1.20	0.96	1.71	17.1	1.0

Use with Soap. More neutral soap is wasted with hard water than with one containing an excess of borax or other alkali. According to Whipple ("Value of Pure Water") 1 lb. of average soap will soften 167 gals. of water when hardness is 20 p.p.m., but only 40 gals. when hardness is 100 p.p.m.

Cost of soap per 1,000,000 gals. = \$10 × hardness (p.p.m.)

This cost of soap is a measure of the depreciation due to hardness. Not only does hard water destroy soap, but even when an excess of soap and alkali is used, it is not entirely suitable for laundry purposes, on account of the precipitation of insoluble calcium, magnesium and iron soaps on the fibers of washed fabrics. When these deposits dry, they turn yellow or even a darker color, particularly if the water also contains iron, organic matter or manganese.

Use in Steam Boilers. Slightly alkaline surface waters of low color* are unexcelled for boiler purposes. Hard waters cause the formation of deposits or incrustations which cause loss of heat or explosions. Hard and acid waters are apt to cause corrosion. Waters containing excessive organic matter, alkalies, (Na_2CO_3) or dissolved neutral salts (Na_2SO_4) are apt to cause foaming and priming.

Table 273. Troubles Due to Bad Boiler Feed Water

Trouble	Cause	Remedy
Incrustation	Suspended matter	Filtration: Blowing off
	Soluble salts	Blowing off
	Calcium, magnesium and iron carbonates	Softening by heat or chemical treatment
	Organic matter	Coagulation and filtration
	Calcium, magnesium and other sulfates	Sodium carbonate or hydrate, barium carbonate
Corrosion	Organic matter	Coagulation and filtration
	Grease, Sugars	Treatment with lime and filtration
	Magnesium, chloride and sulfate	Sodium carbonate
	Acids	Alkalies
	Carbon dioxide and oxygen	Lime, sodium hydrate, heat, aeration
	Electrolysis	Zinc plates
Foaming and priming	Sewage and other pollution	Coagulation and filtration
	Alkalies	Heating feed water
	Excessive sodium carbonate	Barium chloride

* Less than 50 p.p.m.

Classification.—Although it is difficult to draw the line between good and bad waters, the Committee of the American Railway Engineering & Maintenance-of-Way Ass'n has given an approximate classification of waters for boiler use (Proc. Am. Ry. Eng. & M. of W. Ass'n, **9**, 134 (1908):

Table 274. Classification of Boiler Waters According to Incrusting and Corroding Constituents

P.p.m.	Classification
0 to 70	Very good
70 to 150	Good
150 to 250	Fair
250 to 400	Bad
400 and above	Very bad

Benefits of softening boiler waters are the savings in labor for cleaning boilers, in repairs, in fuel, in depreciation and the more continuous service of the plant and the avoidance of accidents. All waters containing more than 250 p.p.m. of incrusting solids should be treated before use in steam boilers, and waters containing more than 150 parts of incrusting solids should be treated if 50 parts of the same consist of sulfates. The ordinary lime-soda process can reduce the incrusting solids to from 35 to 70 p.p.m.

Table 275. Percentage Losses of Heat Units Due to Scale of Various Thicknesses (Booth)

Thickness, inch	$\frac{1}{64}$	$\frac{1}{32}$	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
Loss, per cent...	2	4	9	18	27	38	48	60	74	90

Foaming in locomotive boilers usually begins when the foaming constituents amount to 1700 p.p.m., while in stationary boilers, the limit may be as high as 7000 p.p.m. Waters containing more than 1500 p.p.m. of foaming constituents should be abandoned, if possible. Foaming is the result of suspended impurities which rise to the surface in a more or less dirty condition, forming scum. Pure water cannot produce foaming. Steam from a boiler which foams is dryer than that from a boiler which primes.

Priming occurs when drops of water entrain with the steam, usually suddenly and violently.

Softening Processes. Water to be softened may be treated with chemicals when either hot or cold. The hot process is more rapid than the cold, and is especially adapted to power plants. For municipalities, the cold process is used exclusively. Substances producing temporary hardness are held in solution by carbonic acid. Addition of lime removes the carbonic acid and precipitates the carbonates held in solution thereby. Sulfates, chlorides and nitrates which produce permanent hardness must be treated with sodium carbonate, sodium hydrate or barium carbonate to soften the water.

The dose of chemicals required to soften water may be determined in several ways. American Railway Engineering and Maintenance of Way Ass'n computes quantity of chemicals required to soften water as follows: (E. R., **55**, 50, 1907).

Table 276. Quantity of Pure Reagents Required to Remove 1 Lb. of Incrusting or Corrosive Matter from the Water

Incrusting or corrosive substance held in solution	Quantity of reagent (pure)	Foaming matter increased (lbs.)
Sulphuric acid........	0.57 lb. lime plus 1.08 lbs. soda ash.	1.45
Free carbonic acid.....	1.27 lbs. lime.	None
Calcium carbonate.....	0.56 lb. lime.	None
Calcium sulfate.......	0.78 lb. soda ash.	1 04
Calcium chloride.......	0.96 lb. soda ash.	1.05
Calcium nitrate........	0.65 lb. soda ash.	1.04
Magnesium carbonate..	1.33 lbs. lime.	None
Magnesium sulfate..	0.47 lb. lime plus 0 88 lb. soda ash.	1 18
Magnesium chloride....	0.59 lb. lime plus 1 11 lbs. soda ash.	1 22
Magnesium nitrate.....	0 38 lb. lime plus 0 72 lb. soda ash.	1.15
Calcium carbonate. ...	1.71 lbs. barium hydrate.	None
Magnesium carbonate	4 05 lbs. barium hydrate.	None
Magnesium sulfate.	1 42 lbs. barium hydrate.	None
*Calcium sulfate .	1.26 lbs. barium hydrate.	None

As a general principle (Stabler, E. N., **60**, 355, 1908) lime should be added in sufficient quantity to provide hydroxyl (OH) to combine with the iron, aluminum, magnesium, bicarbonate and hydrogen ions, and with carbon dioxide. In addition, if the carbonate ion (CO_3) in the water plus that formed by change of bicarbonate ion and carbon dioxide is not sufficient to precipitate the calcium present in the water, and added as lime, an additional quantity of CO_3 must be provided by the addition of soda ash in order that all the calcium may be precipitated. This latter consideration determines the quantity of soda ash to be added. The formula is as follows:†

$$C = 1.12\ \mathrm{Fe} + 3.46\ \mathrm{Al} + 2.56\ \mathrm{Mg} + 30.96\ \mathrm{H} + 0.51\ \mathrm{HCO_3} + 1.42\ \mathrm{CO_2}$$
$$D = 2.00\ \mathrm{Fe} + 6.18\ \mathrm{Al} + 2.78\ \mathrm{Ca} + 4.58\ \mathrm{Mg} + 55.44\ \mathrm{H} - 1.86\ \mathrm{CO_2} - 0.92\ \mathrm{HCO_3}.$$

In the formula, CO_2 = carbon dioxide and H = free acid expressed in terms of equivalent hydrogen. C = the quantity of 90 per cent. lime and D the quantity of 95 per cent. soda ash in p.p.m. required to soften the water.

The quantity of chemicals to be added to the water may be determined directly by experiment according to the method of P. Drawe.‡ 200 c.c. of the cold water to be softened are mixed with 50 c.c. of saturated lime water of known strength in a 250 c.c. volumetric flask and heated to boiling. The strength of the lime water must be determined for each series of tests. After cooling, the flask is filled to the mark with water, the contents are mixed, and 200 c.c. are filtered through a dry filter and titrated in a porcelain dish with $\frac{N}{10}$-HCl with methyl-orange as an indicator.

If "b" c.c. HCl are necessary for this purpose, and "a" c.c. $\frac{N}{10}$§ CaO were

* In precipitating calcium sulfate, there is also precipitated 0 74 lb calcium carbonate or 0 31 lb. of magnesium carbonate, the 1.26 lbs of barium hydrate performing the work of 0 41 lb of lime and 0 78 lb of soda ash, or for reacting on either magnesium or calcium sulfate, 1 lb of barium hydrate performs the work of 0 33 lb of lime plus 0 62 lb of soda ash, and the lime treatment can be correspondingly reduced.

† The elements are determined by quantitative analysis and multiplied by the various factors in the formula. The so determined quantities are then summed.

‡ The Addition of Chemicals for Water Softening, Dr P. Drawe, Zeit f angew. Chem (1910), **23**, 52–54

§ See footnote, p. 790.

contained in the 50 c.c. lime water, the milligrams of lime per liter required to soften the water tested may be computed by the following formula:

$$(4a - 5b)3.51 \text{ CaO}.$$

Then add to the neutralized solution in the porcelain dish 20 c.c. $\frac{N}{10}$-Na_2CO_3 and heat to the beginning of boiling. The contents of the dish are washed with water free from CO_2 into a 250 c.c. flask, cooled, made up to the mark, mixed and filtered. 200 c.c. of the filtrate are measured off and the excess of alkali titrated with $\frac{N}{10}$-HCl. The number of c.c. used is designated by c. Then the number of milligrams of soda per liter required to soften the water treated may be computed by the following formula:

$$\left(20b - \frac{5}{4}c\right)33.13 \text{ Na}_2\text{CO}_3.$$

The results by these formulæ are for pure chemicals. For 90 per cent. lime and 95 per cent. soda corrections should be made. In order to get good results by this method the most careful work and the exact preparation of normal solutions, etc., are necessary. When the hardness is over 350 p.p.m., 100 c.c. of water should be used for the determination. For alkaline waters b in the formulæ is to be diminished by the number d, which equals $\frac{8}{5}$ (carbonate hardness —total hardness). The correctness of the above amounts for the desired effect can be proved only by a practical experiment. For this purpose Drawe takes 1 liter of water, adds the calculated amounts of dry lime and soda and heats the whole to 70° C. Water treated in this way as a rule has a slight soda alkalinity and a hardness of less than 18 p.p.m.

Barium carbonate may be used to treat water high in sulfates. By suitable apparatus, the barium carbonate is kept suspended in the water and becomes dissolved only in proportion to the sulfates contained in the water.

Permutit Process. "Permutit" ("permutare," to exchange) is the trade name of an artificial zeolite introduced by L. Gans, of Berlin, in 1906. Sodium permutit is made commercially by fusing quartz or feldspar, kaolin and sodium carbonate in an electric furnace and afterward treating the product with water. It is marketed in granular form having particles from 0.5 to 2.5 mm. in diameter. The reaction between hard water and permutit may be illustrated by the following:

$$\begin{bmatrix} \text{CaCO}_3 \\ \text{MgCO}_3 \\ \text{CaSO}_4 \\ \text{CaCl} \\ \text{Etc.} \end{bmatrix} \begin{matrix} +\text{NaAl(SiO}_3)_2 = \\ \text{(Permutit)} \\ \\ \\ \\ \end{matrix} \begin{bmatrix} \text{Ca} \\ \text{(Mg)} \end{bmatrix} \text{Al(SiO}_3)_2 + \begin{bmatrix} \text{Na}_2\text{CO}_3 \\ \text{Na}_2\text{SO}_4 \\ \text{NaCl} \\ \text{Etc.} \end{bmatrix}$$

Hardness may be decreased to zero, but soluble sodium salts will be increased proportionally. After a period of use, permutit becomes saturated and must be regenerated by treating with a 10 per cent. solution of sodium chloride, when the original action is reversed, Na replacing the Ca and the Mg.

Sodium permutit is slowly decomposed by free carbonic acid. To obviate this, the water is first passed through a layer of marble. This adds to the hardness to be removed by the permutit. The device for using permutit resembles a pressure mechanical filter with permutit in place of sand. For convenience, the brine tank is placed above the filter. Prices quoted for sodium permutit vary from 18 cts. to 30 cts. per lb. More than 5 per cent. of the permutit per annum disintegrates and washes away. For a continuous supply duplicate filters are required, one being in service while the other is being regenerated. Compared with the lime-soda process, softening by permutit is expensive, but for dyeing and other purposes requiring perfectly soft water, it is useful. Where the hardness is excessive, permutit may be used as a supplement to the lime process. In any case, care must be taken to remove the suspended matter, especially hydrates, which would accumulate on the surface of the permutit and prevent the desired interchange of bases. It is stated that 7 to 8 times as large quantity of salt as the quantity of calcium oxide present are required for the regeneration of the permutit. Certain natural zeolites may be used in place of permutit.

Table 277. Efficiency, Cost and Quantity of Materials Required to Treat a Given Quantity of Boiler Water

(Kennicott Co.)

One pound of "carbonate of lime" requires for its precipitation:	
0.56 lb. of lime at ¼ ct. per lb	$0.0014
or 0.80 lb. of caustic soda at 2 cts. per lb	0 0160
or 3.15 lbs. of barium hydrate at 2½ cts. per lb	0 0787
or 2.18 lbs. of sodium phosphate at 4 cts. per lb	0 0872
or 11.92 lbs. of tannin extract, 27 per cent., at 2¾ cts. per lb	0 3278
or 2.28 lbs. of sugar at 5 cts. per lb	0.1140
One pound of "sulfate of lime" requires for its precipitation:	
0 85 lb. of soda ash at 1 ct. per lb	$0.0085
or 1.94 lbs. of sal-soda at 0.65 ct. per lb	0.0126
or 1.53 lbs. of barium chloride at 2 cts. per lb	0 0306
or 1.60 lbs. of sodium phosphate at 4 cts. per lb	0 0640
or 8.76 lbs. of tannin extract, 27 per cent., at 2¾ cts. per lb	0 2409
or 1.68 lbs. of sugar at 5 cts. per lb	0.0840

Coagulation and Precipitation. Many of the compounds formed by the reactions are precipitated in the colloidal condition and coagulated slowly. This is particularly true of calcium carbonate, the chief constituent of many waters. Where magnesium hydrate is precipitated, coagulation is rapid. Coagulation and precipitation are greatly assisted by agitation. For this purpose, mixing channels or stirring devices should be employed to hasten the reactions.

Effect of Excess of Lime. Addition of enough lime to remove the free CO_2 has a marked bactericidal effect, as shown by Hoover at Columbus, where the softening plant removes nearly all the bacteria; also by Houston (Studies in Water Supply). At Columbus the water is treated with excess lime whenever it is extremely muddy or extremely hard. At these times the caustic alkalinity is removed by the addition of sulfate of alumina. Similar results are obtained at New Orleans. In view of the higher efficiency and lower cost of chlorine disinfection, the addition of excess lime solely for sterilization is not warranted.

Boiler Compounds. While a steam boiler is not well adapted for the precipitation of chemicals, boiler compounds have their legitimate use, especially for neutralization of vegetable acids and carbon dioxide. Soda ash or caustic soda would cause some of the scale to precipitate as calcium carbonate rather than as sulfate. All sorts of compounds, such as kerosene, potatoes and tannic acid, have been added to boilers to prevent accumulation of scale; most are worse than useless. Compounds which have given most satisfaction are combinations of soda and tannic acid. Tannic acid has a slight action on the iron and prevents accumulation of scale. A compound consisting of 1 gal. of hemlock extract with 2 gals. of water and 3 lbs. of soda ash may be used with many waters. Hemlock extract costs from 3 cts. to 5 cts. a pound and soda less than 1.5 cts. a pound. Trisodium phosphate (Na_3PO_4) is used with excellent results in many boilers fed with acid waters.

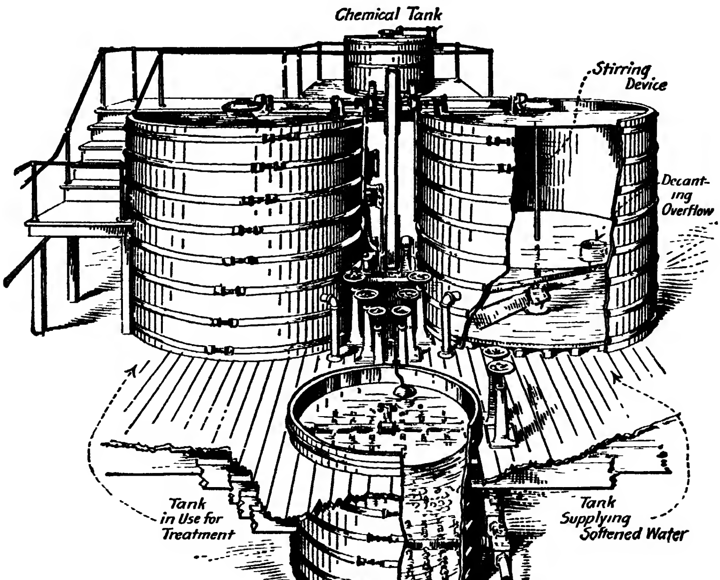


FIG. 371.—Intermittent softening plant.
(We-Fu-Go System)

Types of Water Softening Plants. Softening plants are either continuous or intermittent, and designed to operate with either hot or cold water. In the intermittent system, chemicals are added to water contained in a basin; the whole is mixed, allowed to subside and the treated water is decanted. In the continuous system, chemicals are added proportionally to the flow of water and the treated water is passed through subsiding basin and filter. Fig. 371 illustrates a softening system, operating on the intermittent plan; Fig. 372 a softener, operating on the continuous plan. In the example of latter system illustrated, lime water is prepared in a lime-water tank, or saturator, and

Table 278. Analysis of Scale in Boilers Using Three Different Waters
(Lewes)

Constituents	River water, per cent.	Brackish water, per cent.	Sea water, per cent.
Calcium carbonate	75.85	43 65	0 97
Calcium sulfate..	3.68	34.78	85 53
Magnesium hydrate .	2.56	4.34	3.39
Sodium chloride .. .	0 45	0.56	2 79
Silicon dioxide	7 66	7.52	1.10
Aluminum oxide and iron oxide	2 96	3.44	0.32
Organic matter	3.64	1.55	trace
Moisture	3.20	4.16	5 90
	100 00	100 00	100 00

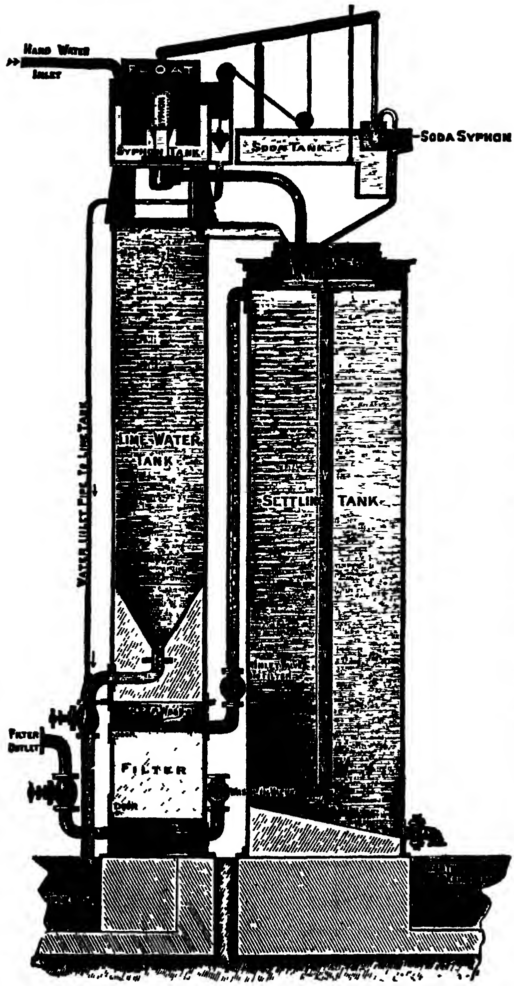


Fig. 372.—Lime-soda water softener.
(Kennicott)

In Fig. 372, the intermittent siphon with accompanying float, lime-feed valve and soda siphon, cause the simultaneous discharge of hard water, lime water and soda solution into the mixing tank. The treated water passes through the settling tank and mechanical filter. Slaked lime is added periodically to the lime-water tank and the portion of hard water diverted through it becomes saturated by the time it is discharged by displacement. Soda solution is added to the soda tank. The bulk of the precipitate is removed from the bottom of the settling tank, the remainder with the wash water from the filter.

added proportionally to the water. Other systems, like the American, Booth, and N. Y. Continental Jewell, add milk of lime instead of lime water. Some manufacturers furnish either milk-of-lime or lime-water apparatus. It is stated by George A. Johnson (U. S. Geol. Survey, W. S. I. paper 315, 1913) that lime water should be used rather than milk of lime, because with the latter the suspended particles of calcium hydrate become coated over with $CaCO_3$

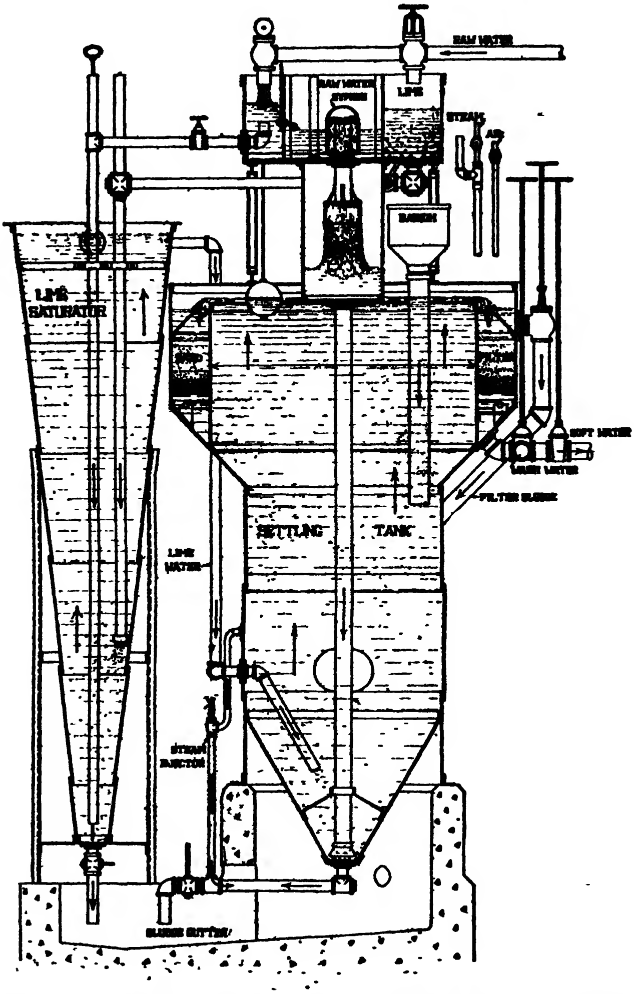


FIG. 373.—Lime-barium water softener, Reisert & Dervaux patents.*
(Sludge from settling tank is discharged into sludge gutter.)

and are rendered inactive. Lime water was first used at Columbus, Ohio. Recently milk of lime has been used. At St. Louis weighed quantities of lime are added to the water at definite intervals of time. The apparatus required to add milk of lime is considerably less costly and cumbersome than that required for lime water.

* Manufacturers and sale agents for U. S., Purification & Eng'g Co., 50 Church St., N. Y. C.

In the *Scaife system*, the feed water first enters the heater, where it is heated to from 200 to 210° F., whereby a portion of the CO_2 is driven off and some of the bicarbonates are precipitated. The chemicals are introduced into the water either before or after heating, and the hot, treated water is passed to a coagulating tank, where the precipitate is removed. The water is then filtered. The Breda system, Brunn-Löwener water softeners, the Pittsburg and many others heat the water during the process.

For the barium process, the apparatus illustrated by Fig. 373 is used. In order to effect the dissolving of the $BaCO_3$ it is necessary to keep it agitated. This is accomplished by the intermittent discharge of the water through a siphon. In general, the plant resembles the lime-soda apparatus. There is a chemical or distributing tank above the main settling tank in which the lime is slaked and run to the raw water, and which also contains the raw-water chamber and the raw-water feeding device. The lime when slaked is run into the bottom of the lime saturator and thereafter a carefully regulated quantity of raw water is passed through the lime and returns through an overflow pipe at the top of the saturator to mix with the raw water in the settling tank in the form of a clear saturated lime water of constant strength. The charge of barium is run into the settling tank at one time for either a 12- or a 24-hr. run. The settling tank contains a centrally located down-take pipe extending very near to the cone-shaped bottom of the settling tank. This down-take pipe connects at its upper end with a chamber in which is located an automatic syphon so arranged that the raw water from the distributing tank is fed to the settling tank intermittently in previously calculated quantities at a time. The effect of this intermittent feeding of the raw water is to produce a regularly recurring pulsation in the water in the settling tank which has the effect of keeping the heavy barium carbonate, which is in a very finely divided state, in continual agitation and brings the barium carbonate into intimate contact with every portion of the raw water. The lime water being continuously fed near the bottom of the settling tank precipitates the bicarbonates of lime and magnesia in the raw water. The barium carbonate, while insoluble in pure water, is soluble in selenitic waters in the proportion that the sulfate radical is present in the water. Therefore, a selenitic raw water will dissolve barium carbonate until its sulfate contents are satisfied. The reaction which takes place then between the barium carbonate and the calcium sulfate produces barium sulfate and lime carbonate, which are both insoluble and are precipitated. This precipitate gradually drops out of the water in its upward course in the settling tank, but to insure the delivery of an absolutely clean water a filter is embodied in the settling tank, which effectually intercepts any precipitate which may still remain in the water. In some plants, in order to prevent the precipitated calcium carbonate from interfering with the action of the barium carbonate, the temporary hardness is removed before treatment with barium carbonate.

Typical Results. At Owensboro, Ky. (Jour. Am. W. W. Ass'n, 1912), 1645 lbs. lime and 16.5 lbs. soda ash were required per million gals. Cost of chemicals \$3.95 per million gals.; total cost including pumping \$6.58 per million gals.; reduction in hardness from 140 to 60 p.p.m.

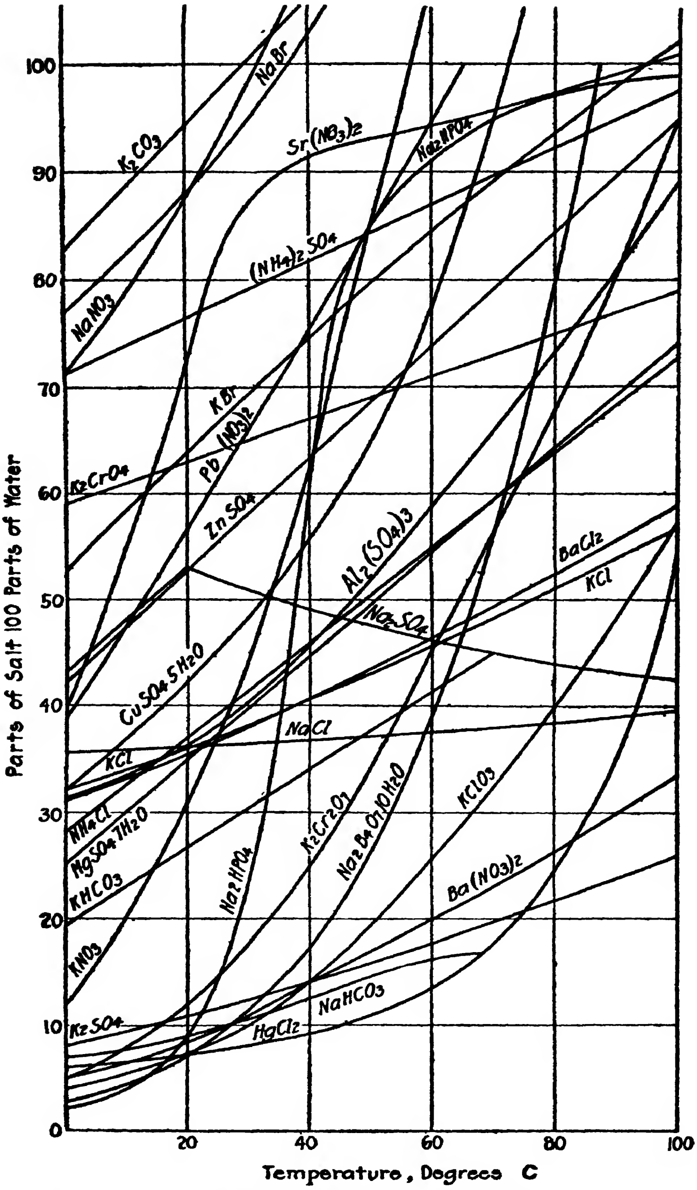


FIG. 374.—Solubilities of various salts in water at various t°C.

For names of chemical compounds, see p. 689.

Table 279. Average Results of Operation of Water Purification and Softening Works, Columbus, O.

HOOVER (Report of 1914)

Average volume of water treated daily.	18,500,000 gals
Chemicals applied, parts per million	
Lime added	164 0
Soda ash added	125 0
Coagulant added.	32 5

	River water	Settled water	Filtered water
Color	18	...	3
Turbidity	54	7	0
Carbon dioxide, free	0	...	...
Carbon dioxide, half-bound	73	...	...
Total hardness	297	95	79
Total alkalinity	166	59	47
Phenolphthalein alkalinity	.	...	25
Caustic alkalinity	0	4	4
Monocarbonate alkalinity		...	43
Incrustants (permanent hardness)	131	35	32
Magnesium	27	...	10
Bacteria per c c	6331	53	14
Chlorine added as chloride of lime	0 14		

All but bacteria expressed in parts per millions.

Table 280. Typical Example of Water Softening in Railroad Practice

Solid constituents (grains per gal)	U. P. R. R , North Platte, Nebraska		C. & N. W. Ry., Council Bluffs, Iowa	
	Before	After	Before	After
Total solids	28 49	25 22	53 67	31.35
Carbonate of lime	7 94	1.98	24 39	2.26
Carbonate of magnesia	1.35	1 08	1.18	0 88
Sulfate of lime	4.53		6 22	...
Sulfate of magnesia	2 43		13.33	...
Silica	2 86	2.01	1 42	0.38
Oxides of iron and aluminum	0.26	0 26	0.34	0 02
Incrusting solids	19.37	5.33	46.88	3.54
Alkali chlorides	3.30	3 24	1.21	1.27
Alkali sulfates	5.82	16 32	5.58	26.54
Alkali carbonates	....	0 33	...	..
Non-incrusting solids	9 12	19 89	6 79	27 81
Lbs. of scale-forming matter in 1000 gals.	2.76	0 76	6 69	0 51

(American Railway Engineering and Maintenance of Way Association, Vol. 5, 1904, p. 590.)

Table 281. Typical Example of Softening of Boiler Waters

(Christie)

Constituents	Grains per U. S. gal. Before	Grains per U. S. gal. After
Oxide of iron (Fe_2O_3)	0.63	0.0
Carbonate of lime ($CaCO_3$)	10.768	1.45
Carbonate of magnesia ($MgCO_3$)	4.777	0 0
Hydrate of magnesia ($Mg(OH)_2$)	0.0	0.87
Sulfate of soda (Na_2SO_4)	0.0	1.802
Sulfate of lime ($CaSO_4$)	1.725	0.0
Chloride of sodium (NaCl)	2.080	2.000
	19.980	6.122

The lime-barium process effects a better removal of the sulfate radical than the lime-soda process.

Table 282. Comparison of Lime-soda and Lime-barium Processes

(Reisert)

	Raw water, p.p.m.	Lime-soda process, p p m.	Lime-barium process, p p m.
Total residue	351 5	209 6	61 4
Mineral	286.9		
CO_2	73 5		
Silica	7.2		
Oxide of iron and alumina	1.0		
Lime	130.5	7.8	8 0
Magnesia	21.4	10 3	9.2
Soda	5.4	77 5	5.4
Sulphuric anhydride (SO_3)	102.4	102 4	4.2
Nitric acid	Trace		
Chlorine	Trace		
Total hardness	285.5	39 3	37 5
Permanent	167.7		
Temporary	117.8		

CHAPTER XXXVI

PRELIMINARY FILTRATION AND DEFERRIZATION

Scrubbers, preliminary filters and contact baffles are filters of coarse material to take the place of, or add to the efficiency of, subsiding or coagulating basins. Fig. 375 shows the contact baffles installed with the subsiding basins at Pittsburgh. These baffles are of concrete, in tank form, divided into units each about 40 ft. by 60 ft., and designed for a filtration rate of 67 mgad. The contact material is gravel from 0.5 to 1 in. in diam., and the depth of material is 8 ft. When clogged, the baffles are cleaned by shutting down one unit at a time, suddenly opening the drain and allowing the water standing in the bed to flush out, carrying with it the accumulated suspended matter. Preliminary filters of this type owe their efficiency in part to plain subsidence and in part to the removal of colloidal matter by contact. For subsidence alone, basins are usually more economical. Contact is of great assistance in coagulation, and in replacing CO_2 and H_2S by oxygen. Preliminary coarse filters may be designed so as to be very efficient for removing suspended matter, even quite fine particles, but are difficult to clean unless of the rapid filter type. Usually preliminary filters, excepting rapid sand filters, filled with material of a diameter smaller than 15 mm. are not practical unless the water contains iron or manganese or both, and little suspended matter.

Puech-Chabal system consists of a series of filters, frequently six, filled with material of decreasing size and increasing depths, and operating at decreasing rates from beginning to end of treatment. This system, utilizing contact to its practicable maximum, also aeration (Fig. 376), is hygienically efficient

Table 283. Characteristics of Puech-Chabal System

Section	Size of material	Depth of material, in	Rate of filtration, mgad.
Roughing filter I........	Less than 20 mm.	12	100 0
Roughing filter II.......	Less than 12.5 mm.	14	50 0
Roughing filter III......	Less than 8.0 mm.	16	30.0
Roughing filter IV.... ..	Less than 5.0 mm.	16	20 0
Prefilter	3.0 mm.	24	10 0
Final filter.	Less than 0.3 mm.	24*	2 5

and gives uniform results, but is costly and not economical. Where water contains colloidal clay ($\frac{a}{t}$ = infinity; see p. 685), the filter effluent cannot be made perfectly clear and colorless unless it contains a natural coagulant like iron. A system consisting of a subsiding basin, coagulating basin and filters, and depending upon coagulation to remove at least the bulk of the suspended matter, is much more economical; and if sterile water be required, chlorine can be applied at small cost.

* Supported on gravel underdrains 2 ft. deep.

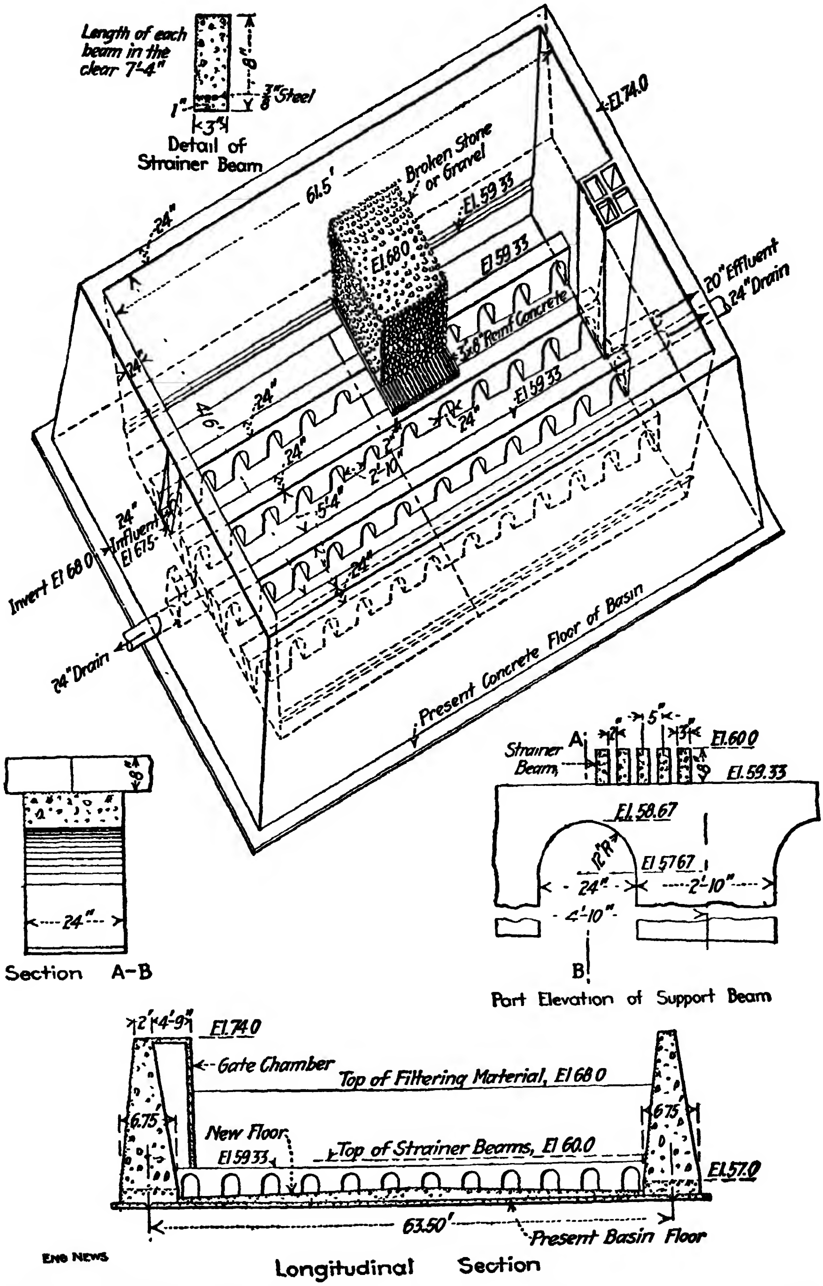


FIG. 375.—Isometric view of typical unit and some details of contact baffles, Pittsburgh, Pa.

(E. N., Oct. 3, 1912.)

Deferrization and Demanganization. Deferrization (from "de" = away from and "ferrum" = iron) is the process of removing iron; *demanganization* that of removing manganese. Iron exists in water as iron hydrate. In the absence of oxygen, this is soluble, unoxidized and usually accompanied by mineral salts, carbon dioxide and other gases, and perhaps by organic matter or manganese, or, indeed, free sulfuric acid. Iron can be precipitated from most ground waters—those low in manganese and vegetable, organic matter—by simple aeration by spraying, followed by filtration through sand or even fine gravel. Provided the water be properly treated beforehand, the kind of filter, whether slow or rapid, whether filled with fine sand or coarse, has little to do with taking out the iron. Simple aeration oxidizes the iron from the soluble unoxidized form to the insoluble oxidized form, from ferrous to ferric hydrate. It also effects the removal of some of the carbon dioxide. One p.p.m. of oxygen will oxidize 7 parts of iron; an excess of oxygen may be obtained by a slight exposure to the air. Ferrous hydrate oxidizes quite rapidly, to ferric hydrate. This latter is insoluble and in most cases coagulates and

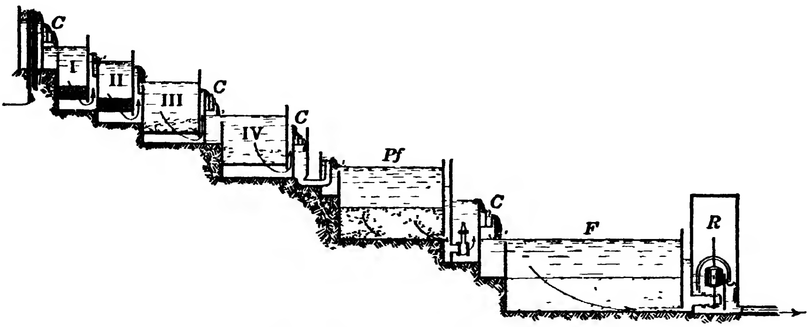


FIG. 376.—Diagrammatic section of Puech-Chabal system.
(I–IV, Dégrossisseurs, *C*, cascades; *Pf*, prefilter; *R*, siphon regulator (Didelon); *F*, finishing filter·
(Don and Chisholm, "Modern Methods of Water Purification," Arnold, 1911)

precipitates rapidly. Some waters contain interfering substances which prevent the precipitation of the iron, holding it in the colloidal form (see "Theory of Colloids" p. 691) and making its removal by filtration difficult. It is not the oxidation but the coagulation of iron which is difficult to accomplish, as is the coagulation of similar small amounts of aluminum, 0.5 to 5 p.p.m. *Acids, organic matter and manganese* all interfere with the precipitation of ferric hydrate.

Pretreatment. Water must be so treated prior to filtration that not only will the iron be oxidized but the carbon dioxide will be eliminated as much as practicable. However, certain soft waters like those at Reading and Lowell, Mass., and West Superior, Wis., which are from alluvial deposits or beneath marshy areas, cannot be saturated with oxygen without causing the formation of a compound of iron and organic acid which is extremely difficult to remove. The presence of a small quantity of carbonic acid seems to be necessary to prevent the formation of this compound. At Lowell, Mass., F. A. Barbour found that best results could not be obtained with the tricklers and filters experimented with if the water were more than 50 per cent. saturated with

oxygen. Certain kinds of organic matter mutually precipitate one another. Other kinds, however, interfere seriously with the deferrization process.

Removal of CO_2. Where complete removal of CO_2 is requisite for coagulation of iron, it can be accomplished by discharging the water over a bed of coke from 2 ft. to 10 ft. thick and at a rate of about 75 mgad. Waters like those at Lowell and Reading can be coagulated and the interference of organic matter avoided by operating the coke bed submerged instead of trickling the water over it. This increases the time of contact.

At *Brookline, Mass., experiments* efficiency of preliminary treatment was found to be dependent largely upon the amount of dissolved oxygen added and of carbon dioxide removed. Table 284 shows the exchange of gases brought about by different methods of preliminary treatment, also the beneficial effect of age due to accumulations of iron hydrate on the surface of the filtering material.

Table 284. Dissolved Oxygen* and Carbon Dioxide* in Brookline, Mass.

Date 1913	Well water		Effluent from devices for preliminary treatment							
			Spray aerator and 9 3-hour coagulating basin		Spray aerator, 2-ft. coke trickler and 1-hour basin		Spray aerator, 5-ft. coke trickler and 1-hour basin		Spray aerator, 10-ft. coke trickler and 1-hour basin	
	O	CO_2	O	CO_2	O	CO_2	O	CO_2	O	CO_2
Aug. 7	1 52	48 0	7 52	21.0	7.62	15.0	9 83	12.0	10 00	8.2
Aug. 21	1 22	41.4	7 87	14 2	9 21	13.4	9.98	10 2	9 36	9.6
Sept. 4	1 25	22 6	6.87	8 3	8.19	6.5	10 57	7.6	9 20	5.8
Sept. 18	1.61	23 4	7 83	11 7	8.67	7 8	9.06	7.5	9.22	5.6
Oct. 2	2.02	24 8	8 04	9.0	8.55	5 9	9 15	5.5	9 30	4.2

* Parts per million.

Effect of Storage. Coagulation could be accomplished, but often at a prohibitive cost, by storing the water for 24 hrs. or more. The plant at Middleboro, Mass. (Jour. N. E. W. W. Ass'n, 1914, **28**, 27)—for outline see Table 295, p. 764—is typical. During 1914, the average results of deferrization were those given in analysis, Table 285. The plant operated a part of each day

Table 285. Results of Deferrization at Middleboro, Mass.
Parts per 1,000,000

Determination	Well water	Subsiding basin effluent	Filter effluent
Turbidity..............................	10.1	7.0	1.6
Color..................................	30.1	17.6	4.6
Oxygen consumed........................	1.98	1.60	1.26
Iron...................................	2.38	0.67	0.20
Manganese	0.71	0.31	0.12
Carbon dioxide.	45 9	5.6	6 4
Dissolved oxygen.	1.20	10.14	9.85

and delivered 320,000 g.p.d. (nearly a third of its possible capacity.) Total cost of operation per mg. delivered to the mains was $3.31. Total fixed charges were $6.02 per mg. Total cost of deferrization was $9.33 per mg.

Fig. 377 shows a typical trickler for use in connection with a deferrization plant. The quick-opening valve connecting with the drain is not shown. The coke is supported on concrete slats and the water is distributed over the surface of the coke by means of a number of sprays. The diameter of the coke varies from 1 in. to 2 in.

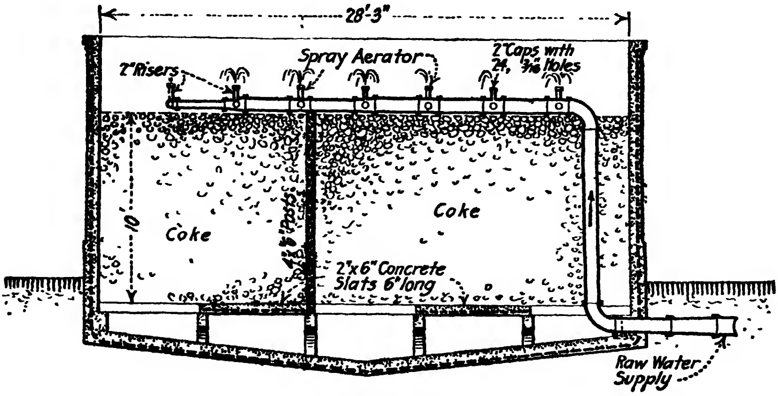


FIG. 377.

Demanganization. Chemically, manganese is closely related to iron; it reacts and precipitates more slowly and possesses the power of preventing the removal of the last traces of iron unless itself be removed at the same time. The process is similar to the deferrization process, but more thorough preliminary treatment is necessary.

CHAPTER XXXVII

FILTRATION

A filter consists of a layer of filtering material, generally sand, through which water is passed for the purpose of removing bacteria and other suspended matter. Said layer is supported by an underdrain system within a vessel or basin provided with various accessories. Filters act primarily as strainers, but the finest particles of silt and the largest bacteria are greatly inferior in size to the smallest sand grains. See Fig. 378. Removal of fine suspended matter, including bacteria, is accomplished in part by the absorptive power of the surface film which surrounds each sand grain, particularly the sand grains in the upper layers of the bed. Another force which assists in the removal of suspended matter is the film which condenses on each particle of suspended matter. These films tend to coalesce with those surrounding the sand grains. See Fig. 379. It was formerly thought that the surface film layer, or "*schmutzdecke,*" accomplished the removal of all the suspended matter, but experience has shown that thicker beds are steadier in operation; it is now believed that each particle of sand plays a more or less important rôle in the removal of suspended matter, although the greater part of the suspended matter is always removed in the surface layer of sand.

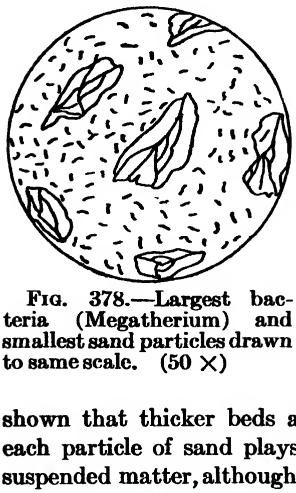


FIG. 378.—Largest bacteria (Megatherium) and smallest sand particles drawn to same scale. (50 ×)

The two principal classes of filters are the *slow* ("slow sand" or "English" or "sand") and the *rapid* (or "rapid sand" or "mechanical") filters.

Slow sand filters differ from rapid filters in that the rate of filtration is much lower and the filter is cleaned by scraping off the superficial layer, which is washed and replaced, either immediately or at some convenient time. Filters washed by the Blaisdell machine differ from ordinary slow filters chiefly in the method of cleaning. For rapid filters, see p. 748.

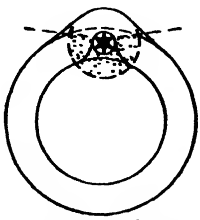


FIG. 379.—Coalescence of films on large and small particles. (Pennink).

Choice of System. For a water having a turbidity less than 30 p.p.m. or a color less than 20 p.p.m., slow filters without coagulation give excellent results. For waters having a turbidity of more than 50 p.p.m. or a color of more than 30 p.p.m., mechanical filters give unquestionably better results. They not only produce an equally safe water but

one of far better appearance. Between these extremes is a region where either the mechanical filter or the slow filter with coagulants may be used equally well. Under ordinary conditions, the latter is far more expensive than the former. It is better for the public health to produce a water of good appearance, even if disinfection has to be resorted to, than to deliver a safe water of so bad an appearance that consumers will resort for drinking purposes to other sources of unknown purity. Well-designed and well-operated filter plants can be depended upon to furnish an agreeable and safe water under practically all conditions.

Intermittent filters are demanded where the water contains so much organic matter that the treatment must approach that required for sewage. They

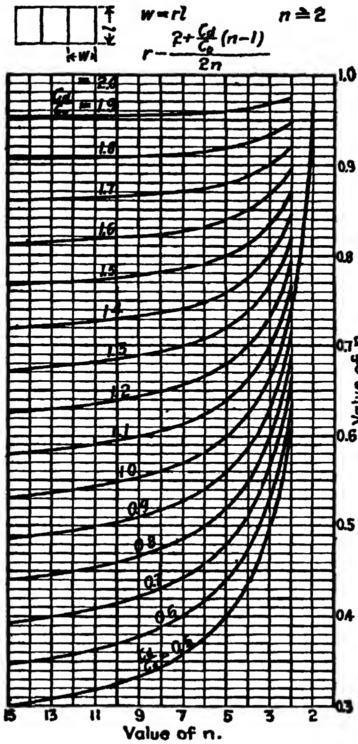


Fig. 380.—(See p. 736.)

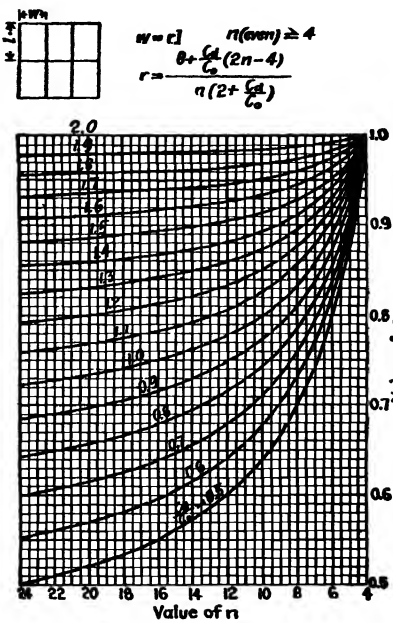


Fig. 381.

are especially necessary for treating odoriferous waters such as exist in the tropics and in certain shallow reservoirs in the temperate zone. Such waters are difficult to purify, either by slow or mechanical filters of the ordinary type. *Double filters* are also used. (*e.g.* South Norwalk, Conn.)

The *population* in the United States *supplied with filtered water* in 1914 (Johnson, Jour. A. W. W. Ass'n, 1914) amounted to 17,291,000, of which 11,893,000 were supplied from rapid and 5,398,000 from slow sand filters. It is estimated that in 1920, 20,000,000 people will be so supplied.

Economical Dimensions of Rectangular Filter Beds. Slow Filters. John H. Gregory (E. N., **44**, 252, 1900) gives diagrams, Figs. 380 and 381, page 735, which are computed from the following data:

n = number of beds.
A = total area of all beds in square feet.
w = width of 1 bed in feet.
l = length of 1 bed in feet.
r = ratio of width to length of 1 bed.
C_o = cost of outside walls per lin. ft.
C_d = cost of dividing walls per lin. ft.
C = total cost of walls.

Fig. 380 applies to a single series, Fig. 381 to a double series of beds.

Number and size of beds in small filter plants are matters requiring judgment more than computation. In large plants, use the method of Gregory (E. R., **47**, 666, 1903).

Area of individual filters depends upon the capacity of the whole plant, and should be small enough so that at least one filter in a small, or two in a large plant can be out of service at all times for cleaning.

Underdrain systems must be so designed that, at the nominal rate of filtration, the frictional resistance of the whole system will be about 25 per cent. of that of the clean sand. In other words, the work should be equally distributed over the whole of the sand area. In large filters, compensating orifices (Fig. 382) may be used in the lateral underdrains in order to effect a better distribution of heads and pressures. Capacities of tile underdrains are given in Table 286; details are illustrated in Fig. 382.

Table 286. Underdrains for Sand Filters (no compensating orifices used)

Rate of filtration, million gallons per acre daily	5	6	8	10	15
Average resistance of clean sand in feet	0.150	0.180	0.240	0.300	0.450
Total allowable friction and velocity head in underdrainage system, feet	0.037	0.045	0.060	0.075	0.112
Approximate ratio of filter area to area of main drain	5100	4700	4200	3800	3200
Approximate maximum velocity in main drain (varying somewhat with size), ft. per sec.	0.90	1.00	1.18	1.34	1.68
Approximate maximum velocity in laterals (varying somewhat with size), ft. per sec.	0.55	0.61	0.72	0.82	1.04

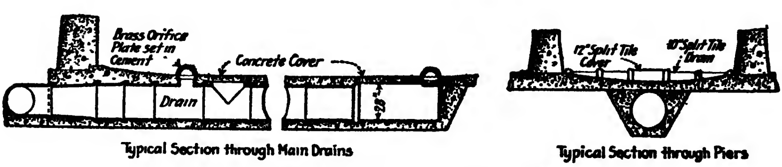


FIG. 382.—Filter drains. (Springfield Mass.)

Table 287. Maximum Areas of Filter Beds Drained in Square Feet *

Diameter of drain, inches	Shape and kind of drain	Rate of filtration, million gallons per acre per day				
		5	6	8	10	15
4	Round lateral	264	245	218	200	168
5	Round lateral . .	420	390	345	316	266
6	Round lateral .	610	570	500	460	390
8	Split lateral.	520	490	430	400	320
10	Split lateral . .	830	770	680	630	530
12	Split lateral	1,200	1,120	1,000	910	770
10	Round main . .	2,700	2,500	2,200	2,000	1,700
12	Round main. .	3,900	3,600	3,200	2,900	2,400
15	Round main	6,200	5,800	5,100	4,600	3,900
18	Round main.	9,000	8,300	7,400	6,700	5,600
21	Round main . .	12,300	11,400	10,000	9,100	7,600
24	Round main . .	16,100	14,900	13,200	12,000	10,000
36	Round main	37,000	34,000	30,000	27,000	22,000

Masonry roofs with earth covers are required for all cold regions, generally speaking, for all cities north of the Potomac and Ohio rivers. South of this line, filters may have to be covered with light-tight roofs to prevent growths of algæ. Typical cross-sections of covered filters at Middleboro and Springfield, Mass., Putnam, Conn., and Washington, D. C., are shown in Fig. 383, which also shows cross-sections of two covered filtered-water reservoirs. The groined arch is usually the cheapest form of roof which affords sufficient head-room above the sand.†

Gravel for slow filters is placed in three or four layers, lowest layer about 7 in. thick of a size passing a 2-in. and retained on a ¾-in. screen, effective size‡ about 20 mm.; middle layer 3 in. thick of a size passing a ¾-in. and retained on a ⅜-in. screen, effective size about 8 mm.; and top layer 2 in. thick, of a size passing a ⅜-in. screen, effective size 2 to 3 mm. The effective size of the supported layer should be at least ⅓ of that of the supporting layer.

Filter sand should be hard and resistant, preferably of quartz or quartzite, and free from excessive quantities of fine particles and dirt of every description. Filter sand should not contain more than 2 per cent. of lime and magnesia calculated as carbonates. It usually requires washing and screening before placing in the filter.

"*Size of sand grains* is determined by sifting through a set of rated sieves. About 110 grams of moist sand are put in a small iron dish and dried in an oven. After cooling, 100 grams are put in the coarsest of a set of sieves, and the sieves are put in a mechanical shaker. A definite number of turns found by experience to be sufficient, is given. Shaking is not continued until no more passes, but only until the amount passing is small, so that doubling the number of shakes would not greatly change the result. The sieves are then taken apart, the material that has passed all the sieves is first put upon the pan of the scale and weighed, then the material remaining on the finest sieve is added to it and again weighed. The process is continued until all the material is on the scale, when it should equal the original weight. The percentages

* Hazen Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley and Sons, Inc., 1916, pp. 931–940. † See page 522. ‡ See page 739.

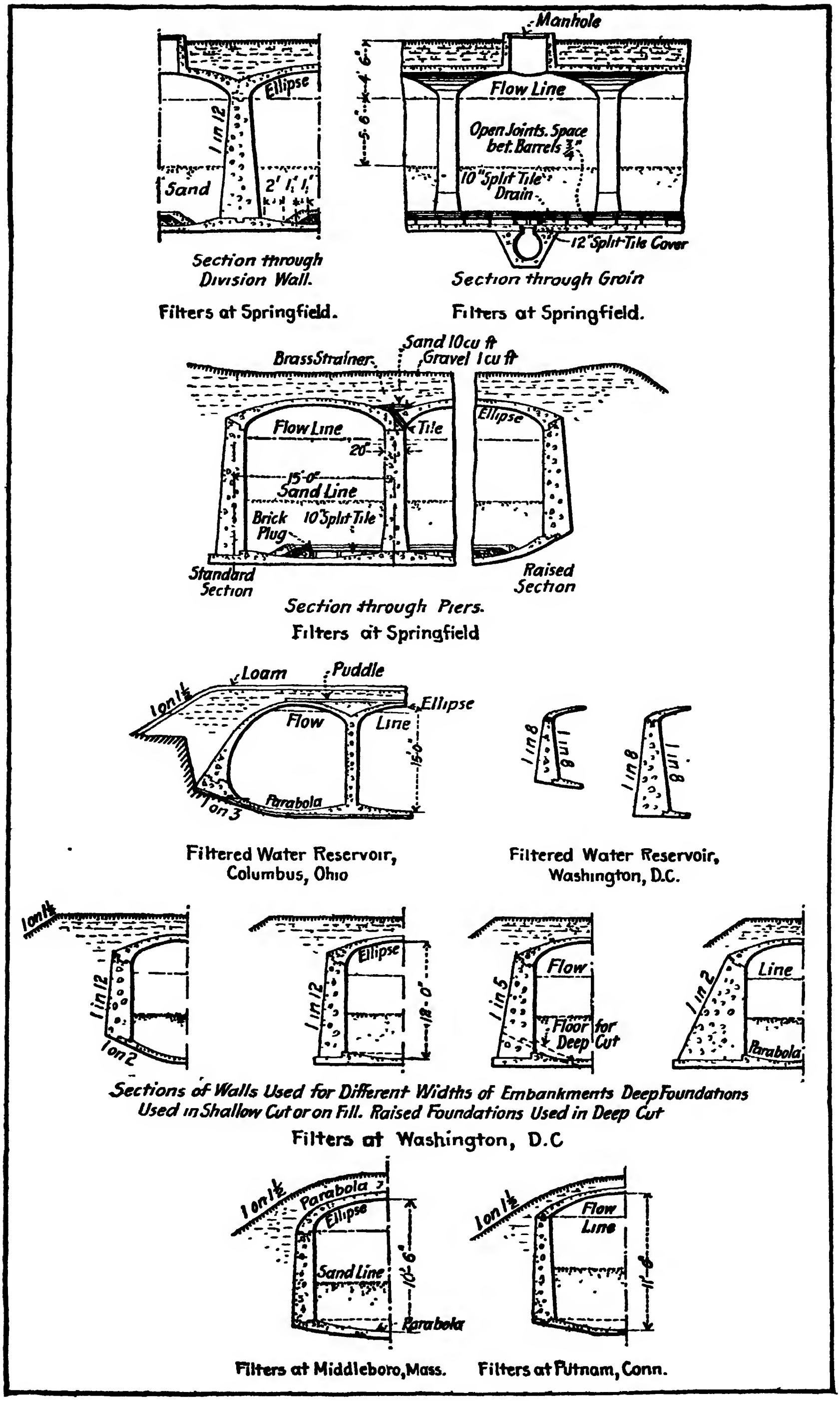

Manhole
Ellipse
1 in 12
Sand
2' 1' 1'
Flow Line
Open Joints. Space bet. Barrels ¾"
10" Split Tile Drain
12" Split Tile Cover
Section through Division Wall.
Filters at Springfield.
Section through Groin
Filters at Springfield.
Sand 10 cu ft
Brass Strainer
Gravel 1 cu ft
Flow Line
Tile
20"
15'-0"
Sand Line
Brick Plug
10" Split Tile
Ellipse
Standard Section
Raised Section
Section through Piers.
Filters at Springfield
Loam
Puddle
1 on 1½
Ellipse
Flow Line
15'-0"
Parabola
1 on 3
1 in 8
Filtered Water Reservoir, Columbus, Ohio
Filtered Water Reservoir, Washington, D.C.
1 on 1½
1 in 12
1 on 2
Ellipse
18'-0"
1 in 5
Flow
Floor for Deep Cut
1 in 2
Line
Parabola
Sections of Walls Used for Different Widths of Embankments Deep Foundations Used in Shallow Cut or on Fill. Raised Foundations Used in Deep Cut
Filters at Washington, D.C
1 on 1½
Parabola
Ellipse
Sand Line
10'-6"
Parabola
Flow Line
11'-6"
Filters at Middleboro, Mass.
Filters at Putnam, Conn.

FIG. 383.

finer than the sizes corresponding to the several sieves are then plotted on a diagram, from which the required data are taken."*

"*Effective size* of sand is the size which is coarser than 10 per cent. of the sand grains by weight. The size of a sand grain is always taken as the diameter of a sphere of equal volume."

"*Uniformity coefficient* is the ratio between the size such that 60 per cent. of the sand is finer than it and the effective size. (See also page 81.)

"In rating sieves an ordinary sand is put upon them and the shaking is performed with the usual number of revolutions. The sieves are then taken apart, each sieve is taken separately, and is given a further slight shaking. A small additional amount of sand passes. The grains so passing are substantially larger than all the grains that have previously passed and smaller than those that remain. This small quantity of sand represents the size of separation of the sieve. A certain number of sand grains are counted out and weighed on an assay balance and the average weight is obtained. The diameter of grain of average size is obtained by the formula

$$D \text{ (in mm.)} = 0.9\sqrt[3]{w} = \sqrt[3]{\frac{6}{\text{sp. gr.} \times \pi}}\sqrt[3]{w}$$

w = weight in milligrams.

There is a little difference in the results of using round-grained and sharp-grained sands, and between grains of different shapes, but the rating is best carried out with various representative sands. Rating of sieves once made does not change appreciably with use until some openings become enlarged or some wires become broken. When this happens the sieves should be at once replaced."*

Sand Analysis. Fig. 384 illustrates the method of analyzing sand and the results obtained thereby; it also shows approximately the relation between certain commercial brass wire cloths and punched metal and the sizes of sand separated thereby. Standard cloths should be used if possible.

"*Turbidity of sand* is the measure of clay in it in p.p.m. Put 10 grams of moist sand into a glass vessel holding 1 liter and fill with clear water. Agitate vigorously until all fine matter is in suspension. Allow to settle 1 min., take the turbidity with a rod and multiply the result by 100. The observed turbidity × 100 = required turbidity. The weight of clay is from $\frac{1}{2}$ to $\frac{2}{3}$ the turbidity, depending upon the size of the clay particles. Turbidity of sand prepared from stock containing clay should always be taken, but when there is no clay in the stock it is unnecessary to do it. Turbidity of slow filter sand should not be allowed to exceed 4000 parts per million, or 0.4 per cent., corresponding to about 0.2 per cent. actual clay."*

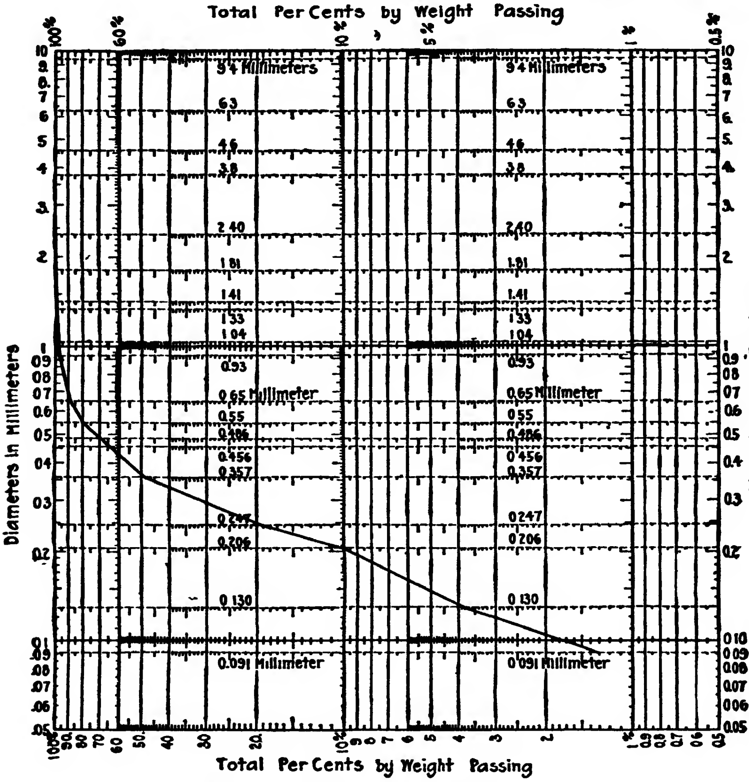
Filter gravel should be prepared by screening and washing. Crushed rock may be used where gravel is not obtainable. Often enough gravel may be sieved from the source of filter sand. Fig. 385 shows the values of the discharge coefficient (c)† for gravel of different effective sizes (p. 741).

Preparation of Filter Sand and Gravel. Sand and gravel for filters can be obtained from a wide range of raw material. Occasionally sand may be

* Hazen Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley and Sons, Inc., 1916, pp. 931–940. † Discharge coefficient (c) for any gravel is 1000 Q (Q = mgad. passing when $\frac{\text{head lost}}{\text{distance traveled}} = 0.001$). Friction head in gravel $= \frac{1}{2}\,\frac{Q \times r^2}{\text{Average gravel depth} \times c}$.

found which is of such size, uniformity and degree of cleanliness that it may be used without screening or washing. Other sands require special preparation. Sand may be dredged from river bars or beaches and pumped through screens or riddles to remove the coarser particles. The finer particles including clay,

Mechanical Analysis of Sand for .... No. 6.250-4'10". Date, June 9, 1910



Mechanical Analysis

Date of Collection, June 9, 1910

Date of Analysis, June 15, 1910

	Size of Sieve	Size M.M.	Am't Passing Grams	Am't Passing Per Cent
Woven Brass Wire	200	0.091	2.6	1.3
	140.	0.130	7.5	3.8
	100	0.206	20.0	10.3
	80.	0.247	38.3	19.6
	60	0.357	94.0	48.2
	Cu 40.	0.456		
	40	0.486	136.5	70.0
	35.	0.55	157.1	80.6
	30	0.65	174.5	89.5
	Cu. 26	0.66		
	25	0.93	188.4	96.7
	20	1.04	191.5	98.3
	16	1.41	193.7	99.2
Punched Metal	0.057"	1.33		
	0.079"	1.81		
	0.103"	2.40	194.3	99.7
	0.156"	3.8		
	0.187"	4.6		
	0.25"	6.3		
	0.375"	9.4		
Total			194.3	99.7
Loss			0.7	0.3
Amount Sieved			195.0	100.0

60% Finer than 0.42 mm.

10% Finer - Eff Size 0.20 mm

Uniformity Coefficient 2.1

Analyst..S.A.W.......

FIG. 384.

may be most conveniently removed by washing the sand in a box deeper at one end than at the other and provided with pipes for distributing the water along its bottom. The dirty sand is dumped at the shallow inlet end and the clean sand is taken out through a sand valve at the deeper outlet end. Mean-

while, the wash water moves upward perpendicularly to the path of the sand and carries with it the dirt and finer particles. Sand containing as high as 10 per cent. of fine material may be prepared by this method. (T. A. S. C. E., **57**, 327, 1906.) Ordinarily the sand for slow filters should have an effective size of from 0.25 to 0.35 mm. and a uniformity coefficient not over 3.0. It should contain not much more than 2 per cent. of calcium and magnesium computed as carbonates. Gravel should be omitted about the piers and along the walls for a width of 2 ft. to lessen the chance of unfiltered water reaching underdrains without first passing through sand. For the same reason piers and walls should be battered or stepped.

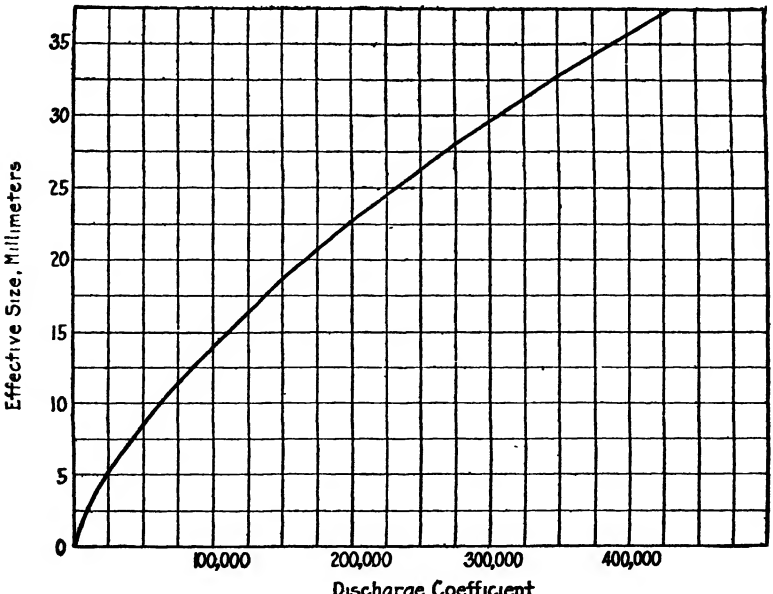


Fig. 385.

Sizes of Separation. The *hydraulic value** of sand of sizes ordinarily used in water filters is approximately as follows: Effective size in mm. $\times$ 100 = hydraulic value in mm. per sec. The size of separation in sand washing boxes depends upon the area of the box and the method of distributing the wash water. The approximate size of separation such that 75 per cent. of the particles of that size will be retained, may be computed by the formula

$$D \text{ in mm.} = 0.065f\frac{\text{gallons per minute of water overflowing}}{\text{square feet of box area}}$$

f = factor determined by size and shape of box, and varies from 3.0 for an ordinary box to 1.5 for a well-designed box with uniform distribution of wash water.†

* Velocity required to float the particle.

† Hazen, Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley and Sons, Inc., 1916, pp. 931–940.

Voids in filter sand vary from 35 to 45 per cent., according to uniformity coefficient and method of placing. Loosely placed sand may settle as much as 15 per cent. after a filter is filled from below with water. Moist, packed sand settles from 4 to 8 per cent. when thus filled with water. Sand placed by hydraulic methods or when perfectly dry, packs closely. *Determination of voids* in sand is made by driving a sheet iron cylinder into the sand so as to fill it completely; the sand removed from the filled cylinder is dried, weighed, and the volume of the solid particles computed (sp. gr. sand = 2.65). The volume of sand is compared with the volume of the cylinder.

Washing and Handling Filter Sand. In all types of filters, sand is usually handled and washed hydraulically. Sand scraped from slow filters is usually

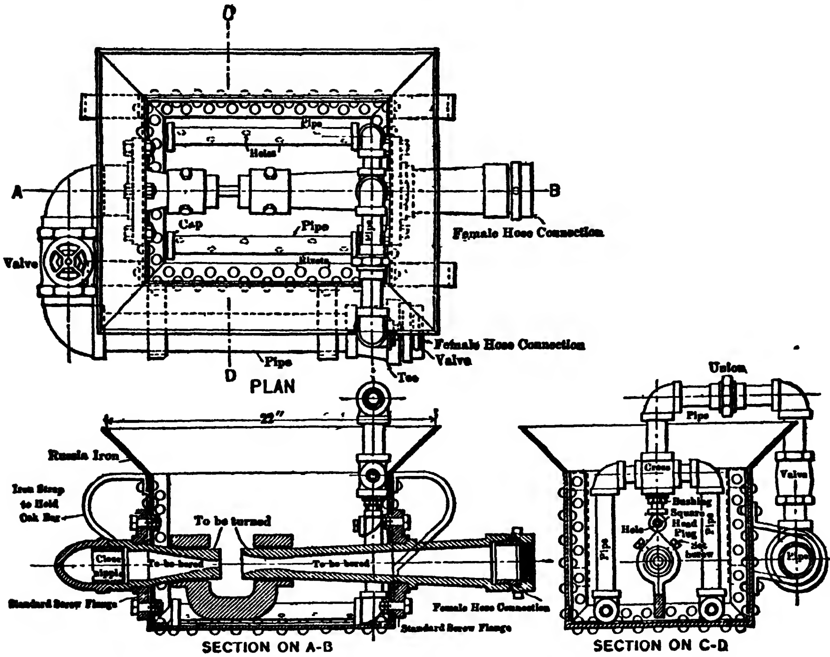


FIG. 386.—Portable sand ejector.
(Stein's "Water Purification".)

gathered into piles and removed from the filter by means of portable ejectors, Fig. 386. Each consists of a tight metal box carrying a large ejector operating with water under pressure furnished through a short 2½-in. hose attached to one of several hose connections in a 3-in. or 4-in. high-pressure main running through the filter. The sand is shoveled into the ejector, usually through a screen to prevent the entrance of gravel into the throat of the ejector. Sand is kept in a suspended state by perforated irrigating pipes in the bottom of the box. It is kept from packing and arching by sprays impinging upon the top sand in the ejector. A mixture of sand and water passing through the throat of the ejector is conveyed through discharge hose and connecting piping to a sand washing machine consisting of two or more hoppers. In these hoppers dirt and fine particles of silt are washed from the sand and then the clean

Table 288. Sand Ejectors and Flow of Water and Sand in Discharge Piping*

Lbs. pressure feed water	Diam. jet, in.	Best diam. for throat, in.	Per cent sand in discharge by volume	Cu. yds sand per hr	Pressure of discharge in ft.	Friction in ft. per 1000 in discharge piping 2½ in	3 in	4 in.	5 in	Lbs. pressure feed water	Diam jet, in	Best diam for throat, in.	Per cent. sand in discharge by volume	Cu. yds. sand per hr.	Pressure of discharge in ft.	Friction in ft. per 1000 in discharge piping 2½ in	3 in.	4 in.	5 in.
60	0.5	0.87	20	5.0	28	150	140		..	100	0.5	0.87	20	6.4	47	162	126	.....	...
	0.5	1.01	25	7.2	20	176	150	.	.		0.5	1.01	25	9.2	34	200	143	...	...
	0.5	1.21	30	10.0	14	206	168	.	.		0.5	1.21	30	12.9	23	250	164	120	...
	0.6	1.04	20	7.2	28	178	124	90	.		0.6	1.04	20	9.2	47	230	133	87	...
	0.6	1.21	25	10.3	20	222	144	110	..		0.6	1.21	25	13.3	34	300	165	103	..
	0.6	1.46	30	14.3	14	275	170	120	..		0.6	1.46	30	18.5	23	380	210	118	105
	0.7	1.06	15	6.5	39	200	114	70	..		0.7	1.06	15	8.4	65	292	144	71	...
	0.7	1.21	20	9.7	28	250	140	88	..		0.7	1.21	20	12.6	47	360	180	88	71
	0.7	1.41	25	14.0	20	325	175	102	..		0.7	1.41	25	18.1	34	470	235	108	83
	0.8	1.21	15	8.5	39	298	145	71	...		0.8	1.21	15	11.0	65	450	208	82	56
	0.8	1.39	20	12.7	28	370	180	89	73		0.8	1.39	20	16.4	47	550	260	104	71
	0.8	1.61	25	18.4	20	480	240	109	82		0.8	1.61	25	23.7	34		340	130	86
	0.9	1.20	10	6.5	52	350	160	61	42		0.9	1.20	10	8.3	87	.	240	78	45
	0.9	1.37	15	10.8	39	435	202	80	56		0.9	1.36	15	13.9	65	.	300	100	60
	0.9	1.56	20	16.1	28	540	250	102	71		0.9	1.56	20	20.8	47	..	370	128	76
80	0.5	0.87	20	5.7	38	154	130	.	..	150	0.5	0.87	20	7.8	71	195	124	90	...
	0.5	1.01	25	8.3	27	186	145	..	..		0.5	1.01	25	11.4	51	240	150	110	..
	0.5	1.21	30	11.5	18	225	160	.	.		0.5	1.21	30	15.8	35	305	180	118	..
	0.6	1.04	20	8.3	38	208	128	90	.		0.6	1.04	20	11.3	71	310	160	86	75
	0.6	1.21	25	11.9	27	260	153	107	.		0.6	1.21	25	16.3	51	400	205	104	83
	0.6	1.46	30	16.5	18	320	187	117	..		0.6	1.46	30	22.7	35	520	260	125	100
	0.7	1.06	15	7.5	52	245	127	70			0.7	1.06	15	10.3	98	410	190	78	56
	0.7	1.21	20	11.2	38	305	158	86	.		0.7	1.21	20	15.4	71	500	236	98	71
	0.7	1.41	25	16.2	27	400	204	104	71		0.7	1.41	25	22.2	51	640	310	120	85
	0.8	1.21	15	9.8	52	370	176	76	56		0.8	1.21	15	13.5	98	..	285	99	59
	0.8	1.39	20	14.7	38	465	222	95	71		0.8	1.39	20	20.0	71	...	350	123	75
	0.8	1.61	25	21.2	27	600	290	115	84		0.8	1.61	25	29.0	51	.	450	160	93
	0.9	1.20	10	7.5	70	450	200	70	43		0.9	1.20	10	10.2	131		335	100	50
	0.9	1.36	15	12.4	52	550	248	91	58		0.9	1.36	15	17.0	98	....	410	129	67
	0.9	1.56	20	18.6	38	680	315	114	73		0.9	1.56	20	25.5	71	.....	510	163	85

* Hazen, Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley and Sons, Inc., 1916, pp. 931–940.

sand is discharged by the ejector at the bottom of the final hopper, to a storage pile in a sand court, to elevated storage bins, through a special device for separating sand from water which can be placed over a filter manhole and discharge the sand directly into the filter, or directly into another filter which is out of service. In the latter case, the sand may be discharged through multiple nozzles at the end of the sand hose supported by a boat or by a carriage supported on tracks within the filter. Meanwhile, water is passed slowly through the filter from below, escaping over the wall of the inlet chamber. The water surface serves as a guide for leveling the sand.

"*The resistance* to be overcome in the sand discharge piping is made up of actual lift and friction. For the lift, multiply the actual lift in feet by the specific gravity of the mixture. For friction of sand and water in 3- and 4-in. pipes, compute the friction for water alone, and add 3.5 ft. per thousand for each per cent. of sand in the mixture. For 6-in. or larger pipe add 2.5 ft. and for 2.5-in. hose, add 4.5 ft. per thousand. These figures will be close enough for velocities 5 ft. per sec. or over. Sand and water mixtures will flow well at all velocities above 5 ft. per sec. and fairly well from 4 to 5 ft. per sec. Between 3 and 4 ft. per sec. there will be more friction than calculated and some stoppages; and below 3 ft. per sec. filter sand and water mixtures will not flow.

$$\left.\begin{array}{l}\text{Velocity in}\\ \text{ft. per sec.}\end{array}\right\} = \frac{137.5 \text{ cu. yds. per hr.}}{\text{Per cent. of sand in discharge} \times (\text{diameter of pipe in inches})^2}$$

$$\text{Cubic yards per hour} = \frac{\text{Per cent. sand} \times \text{velocity} \times \text{diameter}^2}{137.5}\text{"*}$$

Hydraulics of Sand Handling. Theoretically the best form for the throat of an ejector is that which approximates the Venturi tube. Given this shape and the best size for any given addition, the approximate relations are those given in Table 288.

Ejector throats wear rapidly and the efficiencies with somewhat worn throats would be those computed by the formula below (Table 289). Some-

Table 289. Sand Handling Data†

Per cent. of sand in water thrown by vol.	5 0	10 0	15 0	20.0	25 0	30.0
Specific gravity of mixture	1 05	1 10	1 15	1.20	1.25	1.30
Per cent. slush by volume	8 3	16.7	25 0	33 3	41.7	50.0
Per cent. nozzle water by volume	91 7	83.3	75.0	66.7	58 3	50.0
Weight of slush† per part water from nozzle	0.15	0.32	0.53	0.80	1.14	1.59
Q = ratio total weight of discharge to weight of jet water	1.15	1.32	1.53	1.80	2.14	2.59
P = proportion of jet pressure developed in discharge	0.50	0 37	0 28	0.20	0.14	0.10
T = ratio of diameter of throat to diameter of jet	1.18	1.33	1.51	1.73	2.02	2.43
V = ratio of velocity in throat to velocity in jet	0.79	0 68	0.59	0.50	0.42	0.35

The formula $(P + V)T^{1.5} = 1.65$ applies to well-shaped Venturi throat ejectors throwing sand. The best results are obtained when $QV = 0.9$ with 5 or 10 per cent. variation either way, and within this approximate range $PQ^2 = 0.65$. This is the most convenient equation for comparing efficiencies.

* Hazen, Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley and Sons, Inc., 1916, pp. 931–940.

† Slush = suspension of sand in water.

times throats are lined with soft rubber to lessen wear, or readily replaceable wrought-iron nipples are used.

Nichols sand separator, Fig. 387, is a device which combines the functions of both hopper-washer and sand separator. Sand and water are conveyed to this machine by means of an ejector (see p. 742). As the mixture, containing from

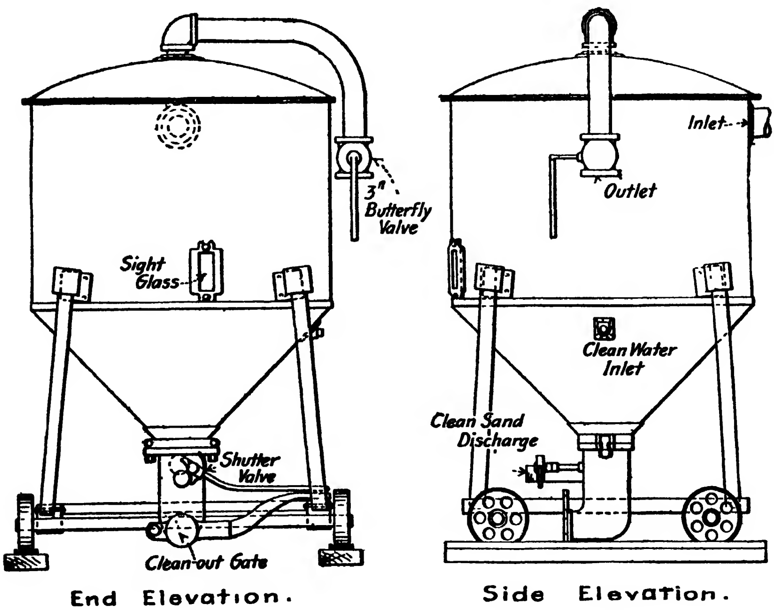


Fig. 387.

10 to 25 per cent. of sand, passes into the separator, it strikes a series of baffles which precipitate the clean sand to the bottom, where it may be discharged through a valve, while the dirty water discharges through a pipe at the top. The sand is returned to another part of the filter; any depth may be washed.

Table 290. Sizes and Capacities of Nichols Sand Separators

Size	Maximum capacity per hour	Height	Extreme diameter of frame	Weight	Pipe connection
30 in.	7 cu. yds.	5 ft. 0 in.	3 ft. 8 in.	500 lbs.	3 in.
36 in.	10 cu. yds.	5 ft. 6 in.	4 ft. 0 in.	600 lbs.	3 in.
42 in.	15 cu. yds.	6 ft. 4 in.	4 ft. 6 in.	700 lbs.	3 in.

Blaisdell washing machine, Fig. 388, is employed to clean the sand in a slow filter by methods which are analogous to those used for washing mechanical filters. It consists essentially of a steel frame-work carrying an inverted box which moves over the surface of the sand and is sealed thereto by its special form of construction, a specially constructed vertical rake, propelling machinery and pumps. The rake extends the full width of the sand bed. The frame-work traverses the length of the filter on rails, carrying with it the washing apparatus. The 2 sets of motor-driven reciprocating hollow teeth of the rake

with water jets at their ends, agitate and wash the sand as the machine slowly traverses its surface. At the same time, a suction pump removes the dirty water from within the box and discharges into the wash water conduit, which is usually built into the top of the division wall of the filters. The Blaisdell machine permits the use of much finer sand (0.18 to 0.25 mm.) than can be employed to advantage in slow filters cleaned by the usual apparatus; it permits a higher rate of filtration (up to 15 mgad.) without increasing the cost of sand cleaning to a prohibitive degree. It also permits the use of slow filters with uncoagulated waters having turbidity higher than 50 p.p.m. For most advantageous application, it requires long rectangular beds for which the practicable maximum width is probably not in excess of 95 ft. Machines are built with two or more traveling speeds, any one of which may be used at

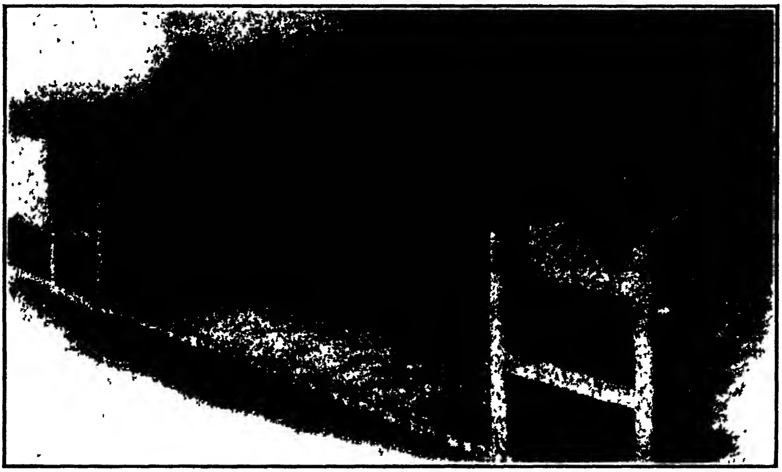


FIG. 388.—Blaisdell washing machine.

the will of the operator. Also the rake may be raised or lowered so as to wash from just below the sand surface or from near the bottom of the sand. The machine at Wilmington, Del.,* is transferred from one filter to another by running it aboard a carriage traveling on rails, perpendicular to the longitudinal axes of the filters. The carriage is then moved opposite the filter where the machine is to be used, and the machine transferred to the tracks along the filter. A tractor machine has been designed for use in slow filters of the ordinary type, traveling on the sand surface instead of being carried on rails by the overhead steel framework.

The cycle or period of a sand filter is the time between cleanings expressed in millions of gallons per acre. That is to say, if a filter operates 20 days at a 5-million rate the cycle is 100. The average cycle for a plant is found by dividing the number of million gallons filtered in 1 year by the total number of acres of filter surface cleaned. Cycles are increased by drawing the water off when the loss of head has reached the allowed limit and raking the surface of the sand and then proceeding. It does not usually pay to rake more

* Plant enlarged in 1916 using ordinary rapid filters.

than once in one cycle. Thorough sedimentation lengthens the cycle. Application of coagulant lengthens it, if applied sufficiently long before filtration, but with a short period of coagulation after application it shortens it. Preliminary filters lengthen the cycle. Cycles commonly range from 50 to 200; if they average less than 50, there is something wrong with the arrangements.* Cycles are virtually the "yields of runs."

Loss of head in sand filters is commonly limited to the depth of water above the sand, 3 to 5 ft., but losses up to 5 or 6 ft. are sometimes permitted, although this is liable to result in "air binding" due to the liberation of dissolved air in the sand.† Initial losses of head with clean sand are given in Fig. 394, p. 752.

Rates of filtration employed with sand filters filtering river waters without preliminary chemical treatment are about 3 mgad. For lake and reservoir waters rates twice as high are used. For filtering river waters after preliminary chemical treatment and subsidence rates as high as 10 mgad. can be used. For deferrization a rate of 10 mgad. is satisfactory.

Table 291. Losses of Head in Feet for Various Rates of Filtration with Clean Sand 3 Ft. Thick

$c = 700 \quad t = 50° \text{ F.}$

Rate of filtration million gals per acre daily	Effective size of sand—millimeters								
	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60
1	0.10	0.06	0.04	0.03	0.02	0.02	0.02	0.01	0.01
2	0.20	0.13	0.09	0.07	0.05	0.04	0.03	0.03	0.02
3	0.30	0.19	0.13	0.10	0.08	0.06	0.05	0.04	0.03
4	0.40	0.26	0.18	0.13	0.10	0.08	0.06	0.05	0.04
5	0.50	0.32	0.22	0.16	0.12	0.10	0.08	0.07	0.06
6	0.60	0.38	0.27	0.20	0.15	0.12	0.10	0.08	0.07
7	0.70	0.45	0.31	0.23	0.18	0.14	0.11	0.09	0.08
8	0.80	0.51	0.36	0.26	0.20	0.16	0.13	0.11	0.09
9	0.90	0.57	0.40	0.30	0.22	0.18	0.14	0.12	0.10
10	1.00	0.64	0.45	0.33	0.25	0.20	0.16	0.13	0.11
12	1.20	0.77	0.54	0.40	0.30	0.24	0.19	0.16	0.13
14	1.40	0.90	0.63	0.46	0.35	0.28	0.22	0.18	0.15
16	1.60	1.02	0.72	0.53	0.40	0.32	0.26	0.21	0.18
18	1.80	1.15	0.81	0.59	0.45	0.36	0.29	0.24	0.20
20	2.00	1.27	0.89	0.66	0.50	0.40	0.32	0.27	0.22
100	10.02	6.37	4.46	3.28	2.51	1.98	1.61	1.33	1.11
125	12.52	7.96	5.57	4.10	3.13	2.48	2.01	1.67	1.39
150	15.03	9.55	6.69	4.92	3.76	2.97	2.41	1.99	1.67
175	17.54	11.14	7.80	5.74	4.38	3.47	2.81	2.33	1.95
200	20.05	12.74	8.92	6.55	5.01	3.96	3.21	2.65	2.23

Owing to high initial loss of head and consequent narrow ranges in losses of head possible, the employment of rates higher than 150 mgad. with sands of effective sizes finer than 0.40 mm. is usually impracticable.

* Hazen Am. C. E. Pocket Book (Mansfield Merriman, editor), John Wiley & Sons, Inc., 1916, pp. 931–940.

† The rate of flow for any loss of head may be computed by Hazen's formula, pp. 78 and 80. C_p = 500 to 700 for sand long in use.

Regulation. Rate of filtration in slow filters may be controlled by hand operated gates and measuring devices preferably of the orifice or Venturi type. These latter may be connected with either indicating or recording devices, the latter preferred. When Venturi meters are used, indicating and recording devices usually show both loss of head and rate of flow. Automatic controllers with hydraulic valves actuated by the Venturi meter mechanism or other controllers may be used but are not necessary for rates less than 6 mgad.

Raking. In some filters, especially those used with deferrization plants, it is desirable to retain as large an accumulation on the surface of the sand as practicable in order to increase the efficiency of the process. In such filters, when the maximum loss of head is reached, the filter is drained, raked and put into service without removing the sand. Raking between scrapings is also practised to reduce cost of sand handling but is not successful with all waters.

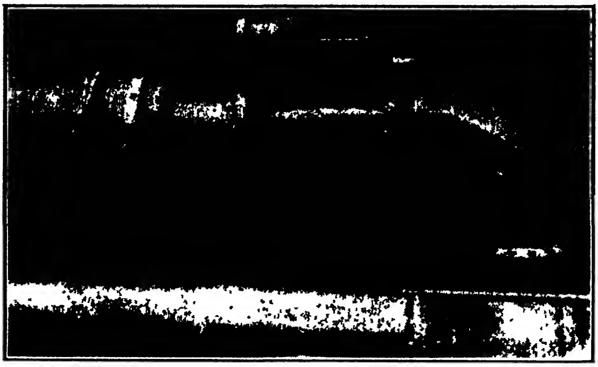


FIG. 389.—Venturi automatic effluent controller.
(Builders Iron Foundry.)

Scraping. When the maximum loss of head is reached, from $\frac{3}{8}$ to 1 in. of sand is removed either by scraping and ejecting as described under "Sand Handling," or, in some old plants, by scraping into wheel-barrows. Sometimes, in cold weather, it is the practice to scrape and pile the sand and remove it from the filter at a more convenient season.

After raking or scraping, the filter should be started slowly, and the rate gradually increased until the efficiency of the filter is established. Filters require a period of biological construction or film-forming in order to attain their maximum efficiencies. Old filters, therefore, are very much more efficient than new.

Per cent. of water for cleaning sand, etc., varies from 0.25 to 1 per cent. of the water filtered.

Rapid, or mechanical, filters are of pressure or gravity type. Fig. 390 shows a sectional view of a pressure filter. These are washed with either raw or filtered water from the pressure main to which they are attached. Fig. 391 shows a typical cross-section of a gravity filter, with mechanical rakes; Fig. 392 of one washed hydraulically at high velocity. Rapid, or mechanical, filters differ from slow filters in that the sand is washed in place by reversed

water currents and the rate of filtration is as great as 50 times that employed in slow filters. It is essential that the concrete work in rapid filters be absolutely tight to prevent the mixture of unfiltered and filtered water, and the entrance of air into the underdrains. The sand usually has an effective size of 0.30 to 0.50 mm.; the uniformity coefficient should not exceed 1.6. Mechanical filter sand should not contain more than 2 per cent. of particles passing a No. 80 sieve (separation size 0.25 mm.). Fine particles in filter sand may be removed by washing in the filter and scraping off the surface.

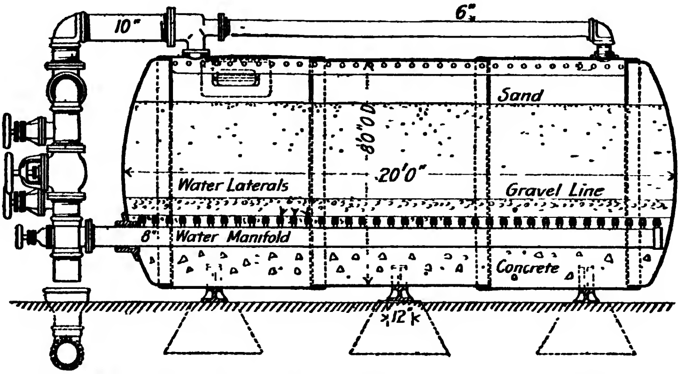


FIG. 390.—Horizontal pressure filter, reverse current wash. (Am. Water Softener Co.)

Arrangement of Mechanical Filters. Ordinarily mechanical filters are arranged in double rows with the pipes in a gallery between. Typical filter pipe galleries are shown in Figs. 392 and 393. In small plants it may be convenient to place two or three filters on one side of the gallery.

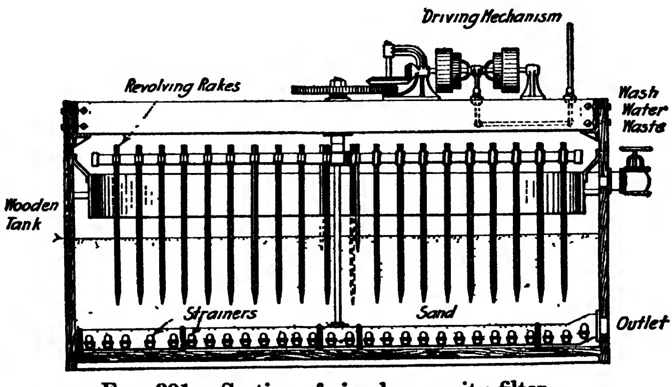
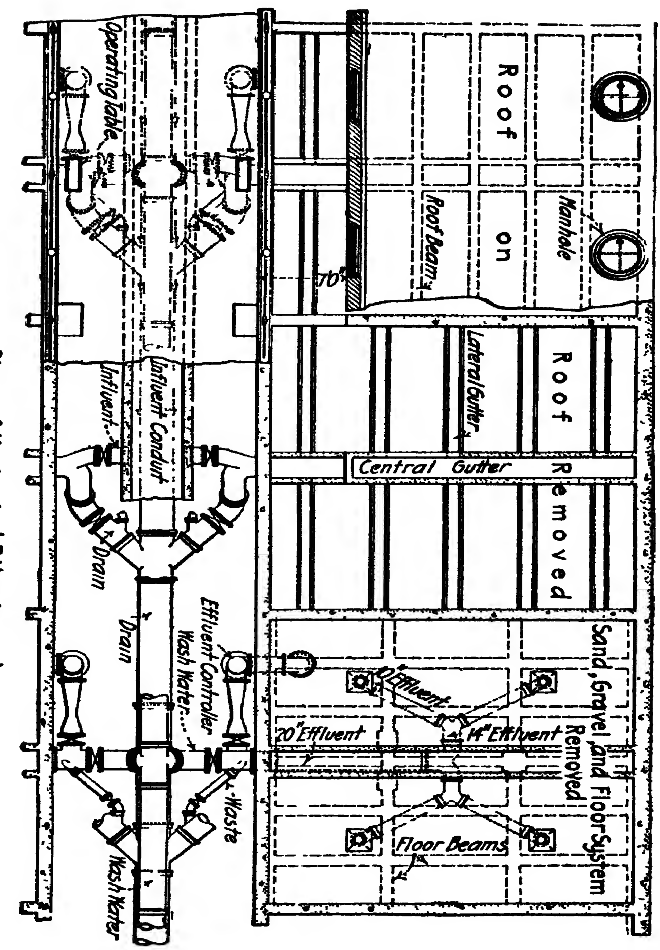
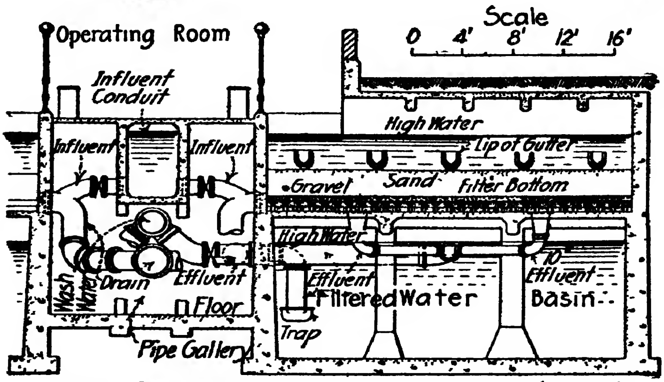


FIG. 391.—Section of circular gravity filter. (Pittsburgh Filter Mfg. Co.)

Areas of mechanical filters per mgd. nominal capacity are varied, but the best practice is to allow for a rate of 2 gals. per sq. ft. per minute plus 5 per cent. or more for wash-water, say an area of 365 sq. ft. per mgd.



Plan of Mechanical Filter (Akron)



Cross Section Through Mechanical Filter (Akron)

FIG. 392.—Akron, O., filter plant.
(Courtesy of F. A. Barbour, C. E.)

FIG. 393.—Typical filter pipe gallery.
(Ithaca.)

Table 292. Areas of Mechanical Filter Units per Mgd.

Location of plant	Engineer	Area per Mgd., sq. ft.	Date
Watertown, N. Y	Hazen	273	1904
Columbus, O.......	Gregory	363	1905
Cincinnati, Ohio...	Benzenberg	350	1907
New Orleans, La..	Earl	358	1909
Clarksburg, W. Va.	Hering & Fuller	300	1911
Evansville, Ind	Alvord & Burdick	368	1912
Evanston, Ill.	Fuller & Pearse	368	1913
Rock Island, Ill.	G. W. Burr	304	1913
Mt. Vernon, Ill.	Severson	352	1913
Cleveland, Ohio	Pratt	348	1914
Baltimore, Md.	Whitman	360	1914
St. Louis, Mo..	Wall	350	1914
Trenton, N. J.	Johnson	347	[illegible]
	Mean =	346	

Friction in underdrain system of mechanical filters, strainers and gravel, varies greatly with the design, but may be computed accurately enough for practical purposes. The relation between *upward velocity of wash-water* and *loss of head* due to presence of various depths of filter sand is shown in Fig. 394.

The strainer system serves for the removal of water and the introduction of wash-water. The friction of the system, when filtering, must not exceed 25 per cent. of the friction resistance of the sand, when the filter is first put into service.

Distribution of wash-water is accomplished by orifices so throttling the discharge that the condition of orifices discharging from a tank is approached.

Where the proportional area of the orifices compared with the underdrains is unduly large, there is a tendency for the sand to be washed out at one place, thus creating a path of least resistance, while in other portions of the bed the clogging is excessive. Continuance of this unequal distribution of wash-water produces *mud balls*, and also greatly reduces the efficiency of the plant. Constants for several filter strainers are given in Table 293. In addition to the orifices or strainers for admitting water, there must be fine strainers or a layer of gravel to retain the sand. Fine strainers are apt to clog and have been quite generally abandoned. Usually the strainers have coarse apertures and are covered with from 3 to 20 in. of graded gravel. In the Harrisburg filters

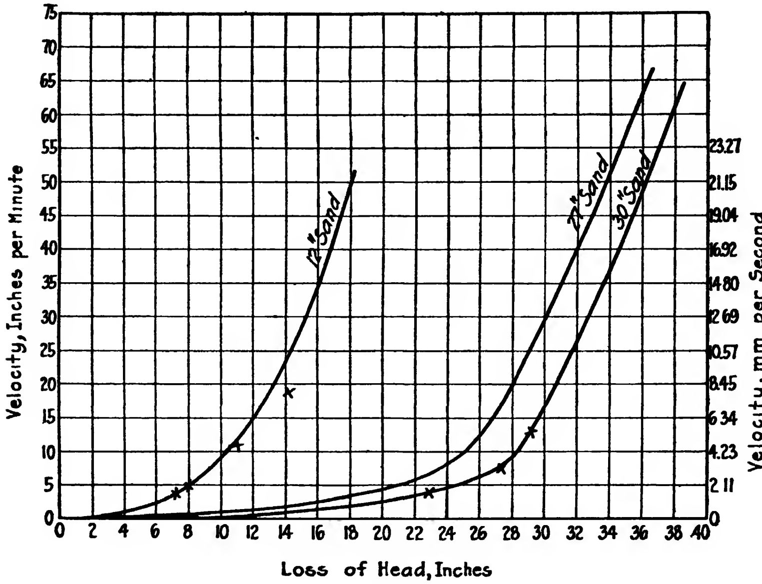


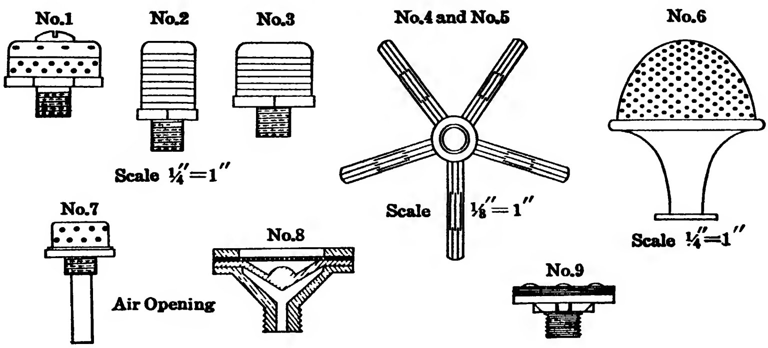
FIG. 394.
(Applies to filter sands of usual sizes.)

(Fuertes) the underdrain system consists of a series of pipes with $\frac{7}{32}$-in. orifices 3 in. apart drilled on their undersides. A 7-in. layer of gravel surrounds these pipes. This method, excellent in other respects, has the disadvantage of exposing the metal pipes to the corrosive action of the water. The *Wheeler filter bottom* is constructed entirely of concrete with the exception of the orifice tube, which is of brass. In place of strainers, a series of five 3-in. balls surmounted by one $1\frac{3}{8}$-in. and eight $1\frac{1}{4}$-in. balls in turn surmounted by a layer of graded gravel from 3 to 6 in. thick is used to retain the sand. Fig. 395 illustrates various types of filter bottoms. The Wheeler filter bottom in the Akron, Ohio, filters (Barbour) is illustrated in Fig. 396, p. 755.

Operating Tables. In large plants, especially, it is convenient and economical to operate all valves electrically or hydraulically. Controllers for these valves are placed on an operating table which also may carry loss-of-head and rate-of-flow gages, devices for sampling, and other devices.

Table 293. Constants for Filter Strainers

		No. 1 Pittsburg Filter Co.	No. 2 Greer Filter Co , A	No 3 Greer Filter Co , B	No. 4 Norwood Eng. Co., A, Brooklyn	No. 5 Norwood Eng. Co., B, Brooklyn	No 6 Paris Water Co., Wm. Wheeler	No. 7 Little Falls, N. J.	No. 8 Louisville, Ky., Jewell (old type)	No. 9 American Water Softener Co., Hodkinson strainer
Diam. connecting pipe	in.	0.5	0.5	0.5	1 0	1 0	Channel	0.5	2 0	0 5
Diam. strainer inlet	in. mm.	0.375 9 52	0 21 5 33	0 4 10 16	0.55 13.97	0 55 13 97	0 5 12 7	0 375 9.52	0 188 4 78	0 22 5 59
Diam strainer holes Width strainer slots	in. mm.	0 067 1 69	0 015 0 37	0.015 0.37	0 008 0 20	0 008 0 20	0.039 1 0	0 067 1 69	0 028 0 71	0 045 1.15
Area strainer inlet ...	sq in sq. cm.	0 110 0 713	0 036 0 229	0 126 0 812	0 238 1 53	0 238 1 53	0 196 1 26	0 110 0 713	0 028 0 180	0 038 0 246
Area strainer holes ..	sq. in. sq cm.	0 104 0 673	0 289 1 86	0 319 2 06	0 515 3 32	0 833 5 37	0 459 2 96	0.101 0 653	0 54 3 48	0 35 2 26
Ratio of area strainer holes to area strainer inlet		0 94·1	8 13 1	2 54 1	2 17 1	3 51 1	2 34 1	0 92 1	19 4 1	9 2.1



Loss-of-head gages may be simple depth gages connected with floats showing the elevation of the water in the chamber connected with the effluent piping. Where the water-level above the filter fluctuates, differential gages are used. Fig. 398 shows a combined indicating and recording loss-of-head gage. Fig. 399 shows a combined loss-of-head and rate-of-flow gage.

Wash-water for mechanical filters is best supplied by gravity from an elevated tank. Where air is employed for washing, it is best supplied from a gasometer. In lieu of storage of air and water in gasometers and tanks, wash-water pumps and air blowers may be used. At Montreal, air is supplied from a gasometer, water from a pump; an ingenious automatic valve shuts off the air supply while the wash-water is running. Pumps should be of a size ample to meet all requirements.

A simple filter for contractors' camps and country use, operating on the intermittent principle, is shown in Fig. 401. This device consists of an in-

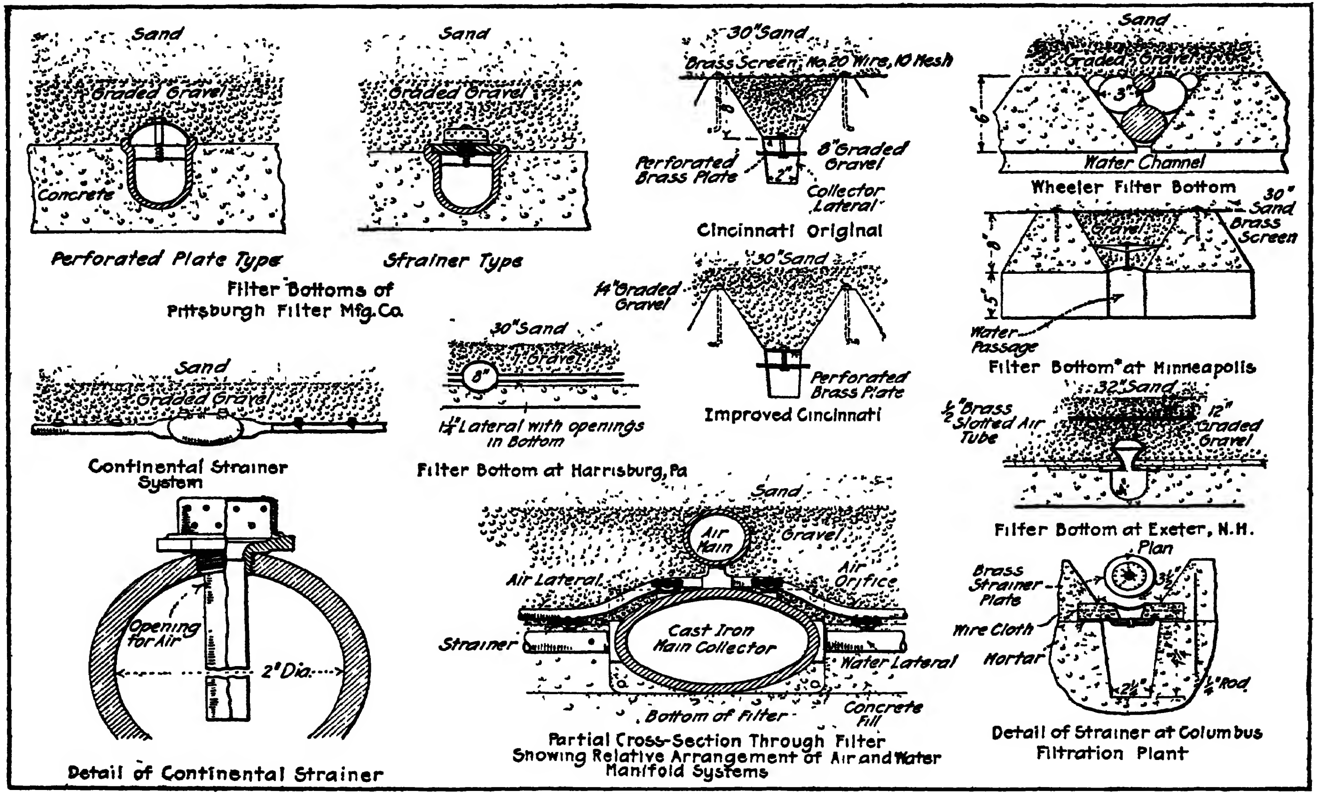


Fig. 395.

* Applies to bridge block over main collector channel.

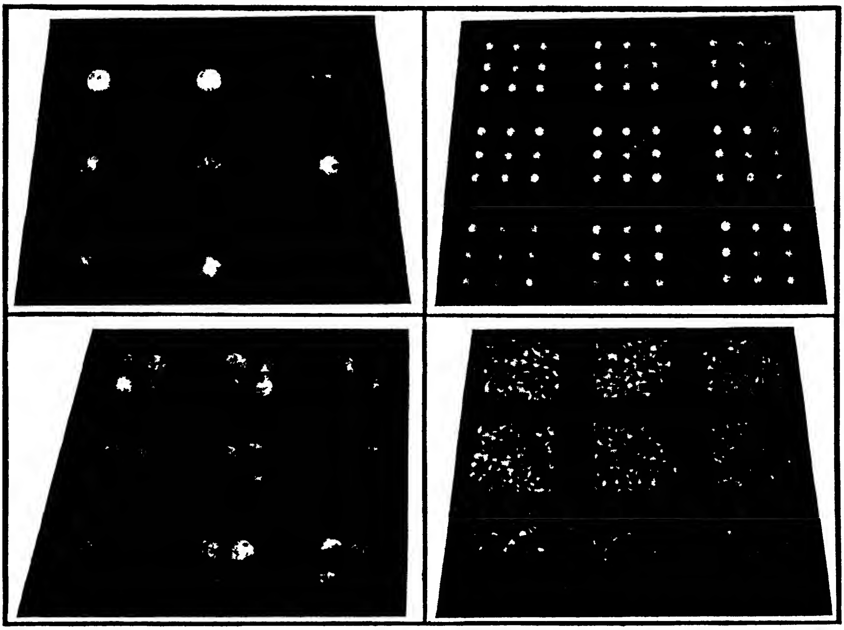


Fig. 396.—Wheeler filter bottom.
Akron, O. (F. A Barbour.)

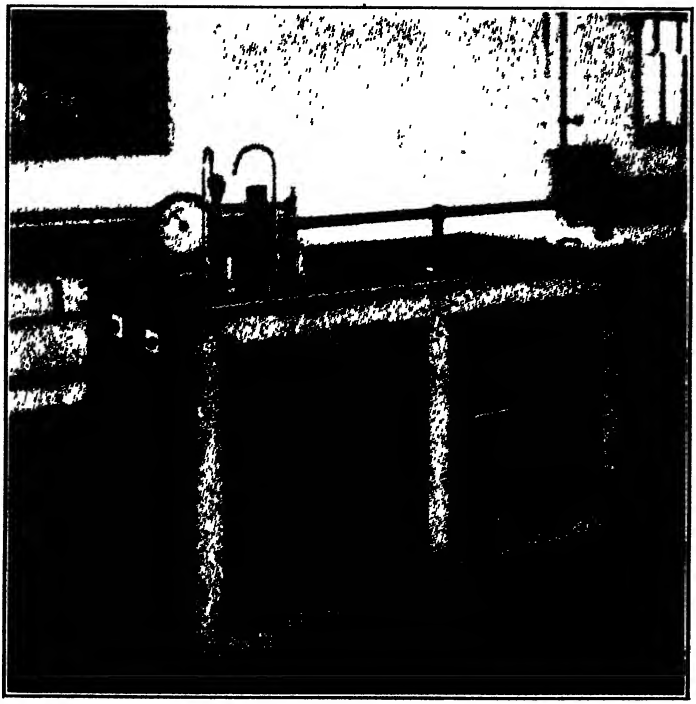


Fig. 397.—View of operating table.

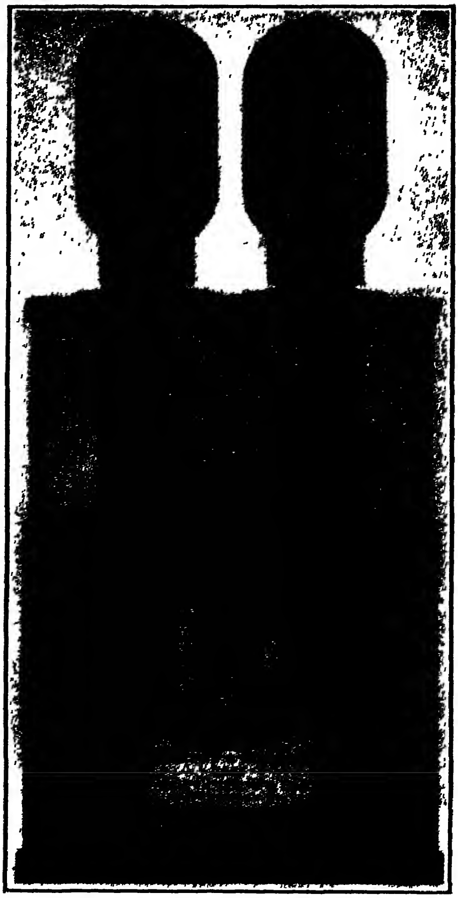
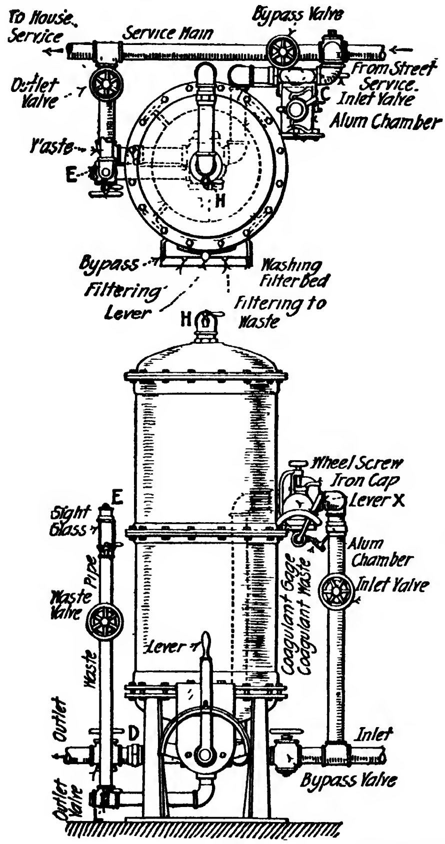


Fig. 398.—Indicating and recording loss-of-head gage. (Pittsburgh Filter Mfg. Co.)

Fig. 399.—Loss-of-head and rate-of-flow indicator. (Builders Iron Foundry.)

termittent sand filter and reservoir made out of a 50-gal. barrel supported on a suitable frame-work. Water is applied to the filter barrel at the rate of three bucket- or garden potfuls at a time. The sand should be allowed to drain after each filling. The apparatus has a capacity of 150 gals. daily. The Newcomb filter, Fig. 402, has been used successfully in the vicinity of Springfield, Mass., for the purification of water containing algæ.



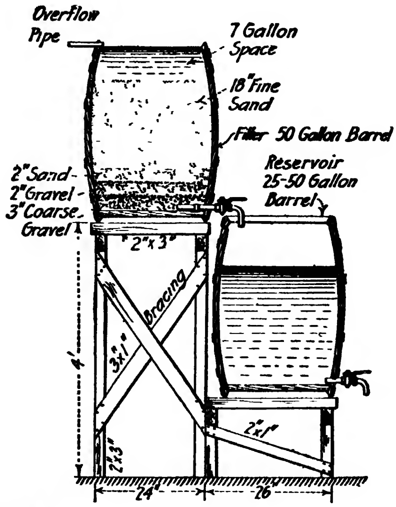


FIG. 400. (Small Filters.) FIG. 401.

FIG. 400.—Front elevation and plan of single-cylinder water filter showing, by shaded lines, suggested by-pass arrangement and connections to be made to supply house and water mains. (Loomis-Manning.)

FIG. 401.—Intermittent barrel filter for drinking water. For contractors' camps, country use, etc.

Rate Controllers. Efficiency of filtration is dependent upon uniform rate of filtration as well as proper regulation of flow of chemicals and thorough mixing with the treated water. Mechanical filters may be controlled by hand and Venturi meters like slow filters, but it is better to use automatic controllers or modules. Of these there are several types. Fig. 403 illustrates a rate controller of the velocity type, combined with loss of head and rate of flow gages. Other controllers employ the principle of ball cock and orifice or of compensating

diaphragm. Fig. 404 shows one of the former, Fig. 405 one of the latter type. Fig. 406 and Table 294 give the dimensions and capacities of *Venturi controller tubes.* The controller may consist of an ordinary hydraulic gate, or valve, actuated by the recording mechanism of the Venturi meter, or by a compensating diaphragm. At New Orleans the filters are controlled by balanced cylinders or weights submerged in, and acted upon by, the water creating the

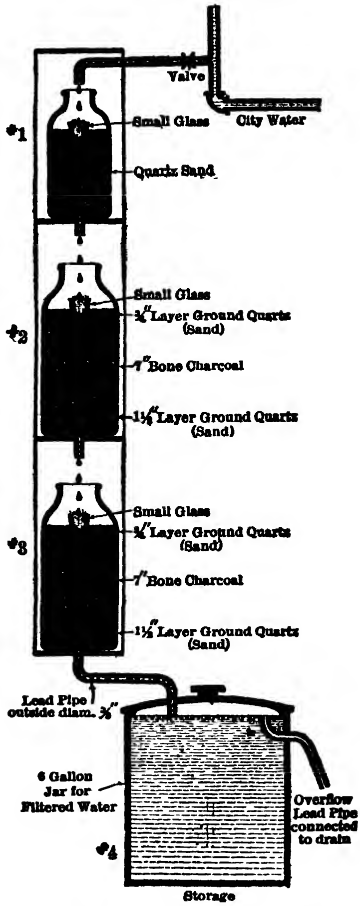


FIG. 402.—Newcomb filter for purifying water for the household.

"A filter of this type, combining aeration and decolorization with charcoal, was much used at Springfield, Mass., in the days of the old Ludlow supply." (Whipple's "Microscopy of Drinking Water," 1914.)

head on the effluent discharging orifice, and by a pressure representing the elevation of water in the equalizing filtered-water reservoir beneath the filters, thereby giving an automatically varying rate depending upon the elevation in the equalizing reservoir, that is to say, an automatic response to the demands of the pumps. The controllers on individual filters may be governed by a *Master Controller* for the whole plant, such as that made by Builders Iron Foundry, Providence, R. I.

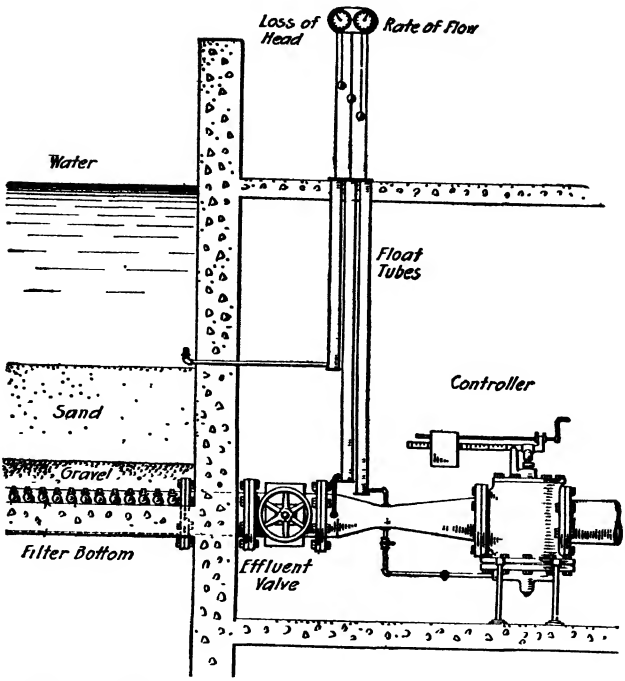


FIG. 403.—Rate controller with combined loss-of-head and rate-of-flow gage for gravity filter. (Simplex.)

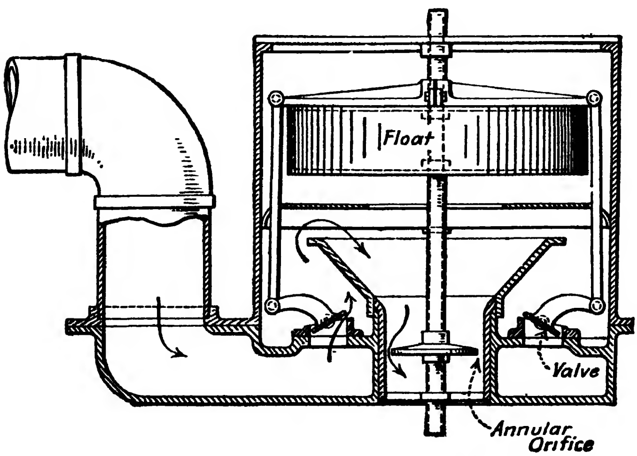


FIG. 404.—Float controller. (Weston.)

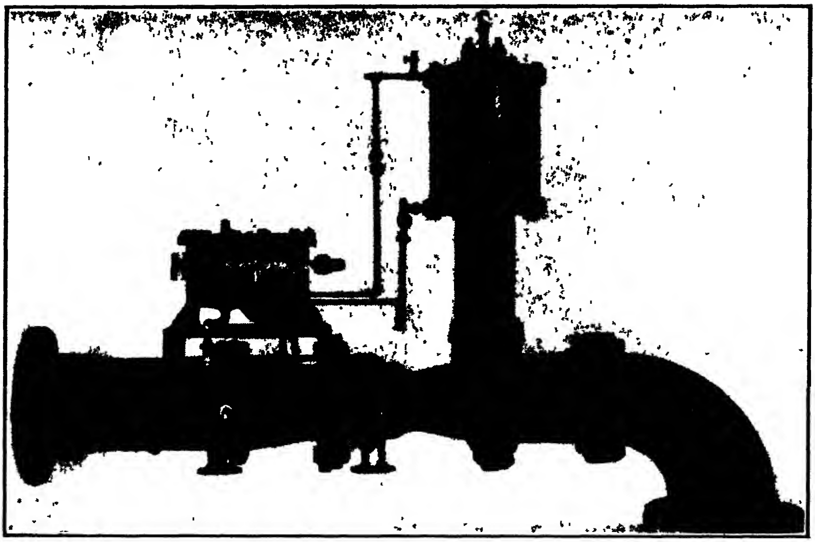
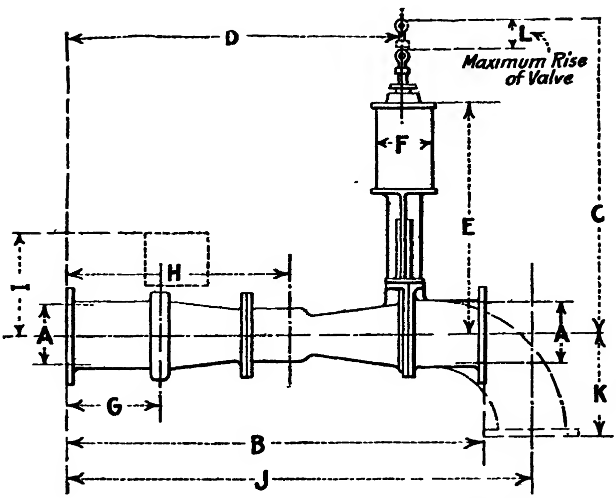


FIG. 405.—Diaphragm controller.
(Builders Iron Foundry)



Dimensions of Venturi Controller Tube.

FIG. 406.

Table 294. Dimensions of Venturi Controller Tube, Inches

A	Minimum capacity, gals. per 24 hrs.	Maximum capacity, gals. per 24 hrs.	Shipping weight, lbs.	B	C	D	E	G	H	I	J	K	L
6	190,000	585,000	450	40	42	32¾	27⅞	11⅛	22¾	16¾	49	10	7⅞
8	338,000	1,040,000	700	50	49	38	33 13/16	12	27	18¼	62	13⅛	8½
10	525,000	1,625,000	1100	60	57	46¾	39½	12¼	31½	18⅜	75	16 9/16	10¾
12	760,000	2,340,000	1550	70	65	54½	44⅝	13	35½	18¾	78	19½	12¾
14	1,033,000	3,185,000	2150	80	72	61⅝	49⅝	13¼	39¾	18¾	92	22⅜	14⅞
16	1,352,000	4,160,000	2900	90	79	69¾	54¾	14	44	19	108	25 7/16	16¾

F = 8″ for A = 6; 10″ for other sizes.

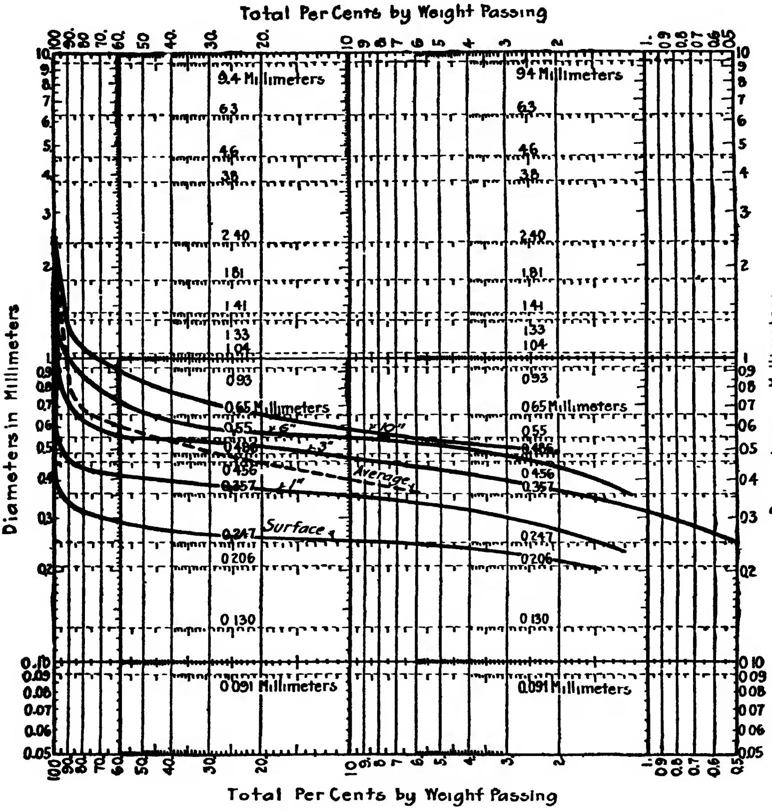


FIG. 407.—Salisbury Beach sand. Summary of analyses of samples from different depths in filter, after washing.

Loss of head in mechanical filters is of course greater than that in slow filters. Ordinarily it reaches a maximum of 8 to 12 ft. at a rate of 125 mgad. When the loss of head exceeds the depth of the filter, the outlet pipe acts as a draft tube and produces suction in the filter. This is known as *negative head.* Negative head has the disadvantage of liberating dissolved air from water and brings about a more rapid clogging of the sand. The full effect of negative head may be utilized by putting an air chamber on the effluent pipe and removing the air by pumping.

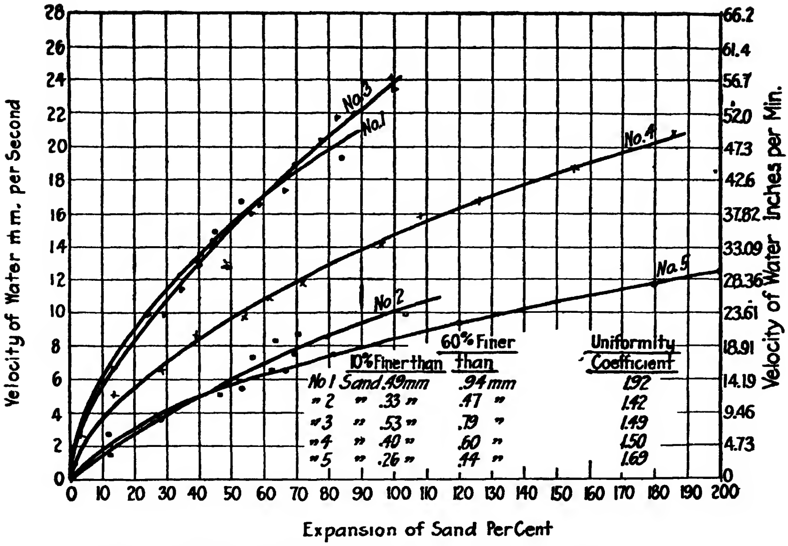


FIG. 408.—Upward velocity of wash water and expansion of sand in mechanical filter.

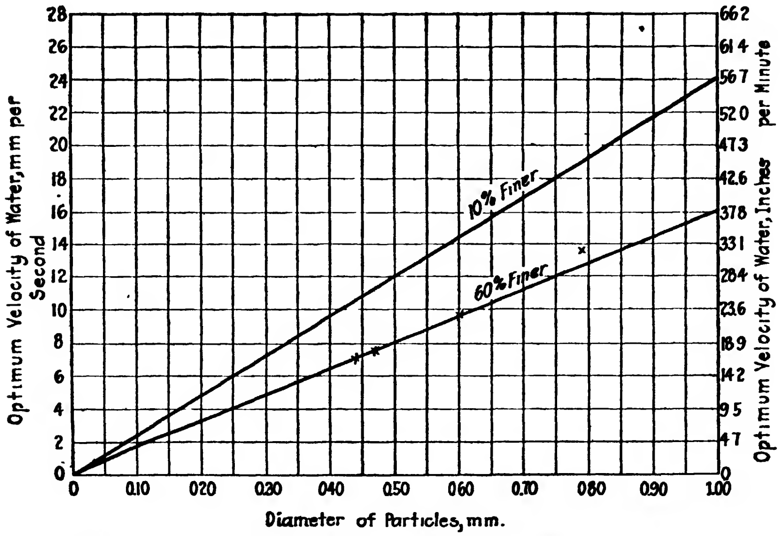


FIG. 409.—Optimum velocities of wash water for sands of various sizes.
Ten per cent. curve is derived from 60 per cent. curve, assuming uniformity coefficient of normal sand to be 1.50.

Example: To determine optimum velocity of wash water and percentage expansion at optimum velocity of a sand having particles 10 per cent. finer than 0.40 mm. and 60 per cent. finer than 0.65 mm. In Fig 409, the optimum velocity is read from the 60 per cent. curve as 10.4 mm. per sec. (or 24.6 in. per min.). In Fig. 410, the degree of expansion is read from the 60 per cent. curve as 50 per cent. The wash-water overflow for a sand layer 30 in. thick should, therefore, to minimize loss of sand, be placed more than 15 in. above the sand surface. Ordinarily the 60 per cent curve is safe, but 10 per cent. curve may be used when uniformity coefficient is 1.5. With a given velocity of wash water, sands with rounded grains expand less than others (Fig. 408).

Washing mechanical filters is done by reversing the current of water through the sand. The most efficient washing is accomplished by *wash-water velocities* of 1.5 or more vertical ft. per min., depending upon size of sand grains. Where these rates are not employed, the use of air or some mechanical device for agitating the sand is necessary for best results. Wash-water should be applied gradually at first and shut off gradually when washing is finished in order to avoid mixing sand and gravel. The sand in mechanical filters, after washing, should be stratified in layers according to the hydraulic values* of the particles, as shown in Fig. 407. This result cannot be obtained unless the sand be thoroughly floated during washing. The degrees of expansion of five different sands for different velocities of wash-water, are shown in Fig. 408.

Mud balls are accumulations of dirty sand within the filtering layer. They never exist where the washing devices are properly designed and operated. Where washing devices do not prevent the formation of mud balls, the latter must be removed by raking or spading the sand by hand during washing.

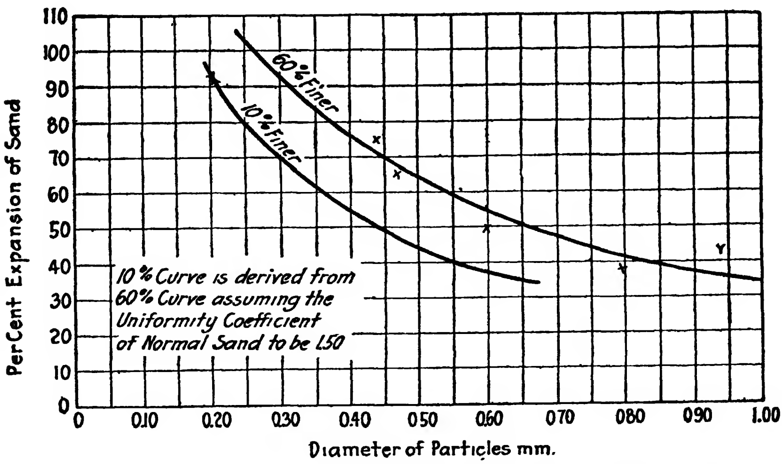


FIG. 410.—Expansions of normal sands at optimum velocities of wash water.

Frequency of washing depends upon character of raw water and degree of removal of suspended matter effected in the coagulating basins. A filter is usually washed when the loss of head reaches a certain maximum, 4 to 10 ft., usually once in 4 to 24 hrs. The coagulum will "break" through in some filters operating with some waters before the maximum loss of head is reached; such filters should be washed immediately, or before breaking.

Duration of washing is usually 4 to 8 min. and the *period out of service for washing* is usually 8 to 15 min.

Quantity of Wash-water. From 1.5 to 8 per cent. of the water filtered is required for washing. Filters should not be washed too clean, otherwise low efficiency will occur right after washing. With non-uniform sand washed at

*See footnote, p 741.

high velocity, the surface of the bed after washing will be composed of very fine particles which may form a layer so dense as greatly to decrease the period between washings. This condition may be overcome in ordinary cases, by scraping off the fine layer after washing; in extreme cases, by so manipulating the wash-water valve that stratification near the surface will not be marked. When the filter clogs frequently, the wash-water may be applied for a short period just to lift the upper portion of the sand bed and without wasting any of the wash-water. Agitation with air may also be used to break up the accumulation of fine sand at the surface of the bed.

Optimum velocity of wash-water (*i.e.*, best velocity for efficient and economical washing) for average sand of different sizes is given in Fig. 409, p. 762.

In studying the relation between *washing velocities* and *filter sand*, the 60 per cent. separation size is of more value than the "effective size" (10 per cent. separation size). Relation between *sizes of sand particles* and *per cent. of expansion of sand layer*, for normal sands, at optimum velocity of wash-water is shown in Fig. 410.

Table 295. Data Relating to Typical Deferrization Plants

Name of plant	Urbana, Ill.	Middleboro, Mass	Iowa City, Iowa
Plant put in operation	1913	1913	1910
Designer. .	A. N. Talbot	R. S. Weston	N. Y. Continental Jewell Filtration Co
Population (1910)	20,666 (including Champaign)	8,214	10,091
Total cost ..	———————	$18,000	$26,000
Cost per million gals capacity.	———————	$18,000	$13,000
Source of supply. ...	Wells	Well (near Nemasket river)	Infiltration galleries in bed of Iowa river
Rated capacity, gals. per day.	2,000,000	1,000,000	2,000,000
Total capacity of subsiding or coagulating basins, gals.	250,000*	40,000	250,000
Total capacity of filtered water basin, gals.	700,000*	42,000	
Chemicals used	Aeration only, no chemicals	Aeration only, no chemicals	Lime
Manner of application of chemicals.	.	....	applied near extrance to settling basins
Number of filter units	4	2†	4
Net area of filter surface, sq ft	720	4,350	700
Depth of filtering materials, in.	Gravel (8), Sand (30)	Gravel (12), Sand (36)	Gravel (12), Sand (28)
Size of filtering materials, mm.	———————	Eff size 0 31, uniformity coef. 1 80	Eff size 0 54, uniformity coef 1 4
Method of cleaning..	Reverse flow of water with agitation by compressed air. Occasionally all of the sand is taken out and washed	Scraped by hand	Reverse flow of water with agitation by compressed air. Vertical velocity of wash-water 1 ft per minute
Number of subsiding or coagulating basins	1	1	2
Control of rate of filtration	Controlled by orifice on raw-water pipe, head being regulated by hand-operated valve	Orifice type controlled by hand-operated valve	Closed type of controller

* These basins were in existence when iron removal plant was built and are used in connection therewith. † Also aerator and trickler consisting of bed of coke 30 ft. diam. and 10 ft deep, see p. 733.

Table 296. Data Relating to Typical Water Softening Plants

	Columbus, Ohio	McKeesport, Pa.	Oberlin, Ohio
Plant put in operation	1908	1908	1903
Designer	J H. Gregory	Alex Potter	C. Arthur Brown
Population (1910)	181,511	42,694	4365
Total cost	Land $48,410 Miscellaneous 76,490 Pumping station (low lift and direct service) 399,240 Purification plant 532,480 Engineering 81,920 Total $1,138,540[1]	$250,000	$15,150
Cost per million gals. daily capacity	$17,750 (purification plant only)	$25,000	$68,900
Source of supply	Scioto river	Youghiogheny river	Vermillion river
Rated capacity (gallons per day)	30,000,000	10,000,000 (softening plant) 6,000,000 (filter plant)	220,000
Total capacity of subsiding or coagulating basins (gallons).	15,000,000	3,300,000	587,000
Total capacity of filtered water basin (gallons).	10,000,000	350,000	None
Chemicals used	Sulfate of alumina, lime, soda ash, bleach	Sulfate of alumina, lime, soda ash	Lime, soda ash
Manner of application of chemicals	Chemicals added to water before entrance to mixing tanks	Lime added at entrance to baffled tank, soda ash just before entrance to settling tank, coagulant after passage of one settling basin	Added to mixing box from which water flows to settling basins
Number of filter units	10	6	2
Net area of filter surface (sq ft)	10,889	2300	77
Depths of filtering materials (inches).	Gravel (10) Sand (30)	Gravel (12) Sand (30)	Gravel (10) Sand (24)
Sizes of filtering materials (millimeters)	Eff size 0.41 Uniformity coef. 1.36	Eff size 0.35 Uniformity coef 2 65	Eff. size (0 39–0.58) Uniformity coef. 1 5
Cleaning of filter system.	Reverse flow of water with agitation by compressed air when necessary	Reverse flow of water with agitation by compressed air	Reverse flow of water and hand raking
Number of subsiding or coagulating basins.	6	4[2]	2
Control of rate of filtration	Controller of closed type—self-contained and requiring no auxiliary devices	Closed type of controller	Hand operation

[1] Includes engineering charge on small amount of other work costing $76,490
[2] When coagulant is necessary, settling basin may be operated in two divisions.

Table 297. Data Relating to Typical Slow Sand Filter Plants

	Philadelphia,[1] Pa.	Pittsburgh, Pa.	Washington, D. C.	Toronto, Ont.	Springfield, Mass.	Albany, N. Y.	Wilmington, Del.
Designer	John W Hill, Chf Eng	Morris Knowles	Allen Hasen	Hasen and Whipple	Hasen and Whipple	Allen Hasen	T. A. Leisen
Plant put in operation	1905	1908	1905	1912	1910	1899	1910
Population (1910)	1,549,008 (whole city)	533,905	343,000	342,000	88,926	100,253	87,411
Total cost	Final filters $7,100,000[3] Pre-filters 1,160,000[3] Intake 225,000 Pumping sta 723,000	Bid on final filters, for original 46 acres = $3,303,000*	Pumping sta. $183,600 Filters & appurtenances 2,197,000 Filtered water resr. 150,000 Eng, land & misc 825,700 Total $3,356,300[5]	Filters $545,900 Pure water resr. 72,600 Pumping sta 90,900 Eng & inspection 72,100 $781,500 Value of land 224,040	Filters and sedimentation basin, $321,700[6]	Pumping sta. & intake $49,745 Filters & sedimentation basin 324,217 Land, eng & conduit 125,928 Total $499,890	Preliminary filters $84,200 Final filters 292,300 Sedimentation basin 175,000 Pumping sta. 308,300
Cost per acre	$142,500 (final filters)	$72,000 (final filters only)	$75,700 (filters and appurtenances)	$56,800 ex. land	$107,200 (inc. sedimentation basin)	$57,900 (inc. sedimentation basin)	$146,000 (final filters)
Source of supply	Delaware river	Allegheny river	Potomac river	Lake Ontario	Little river	Hudson river	Brandywine creek
Rated capacity (gallons per acre per day)	3,000,000 (without pre-filters) 6,000,000 (with pre-filters)	4,000,000	3,000,000	6,000,000	6,000,000	3,000,000[9]	7,500,000
Total cap of subsiding or coagulating basins (gals)	100,000,000 gals	120,000,000[4]	Water flows through 3 resr.'s having total cap. of 649,000,000	None	40,000,000	14,600,000	32,000,000
Total cap of filtered water basin (gals)	50,000,000	50,000,000	15,000,000	7,500,000	17,000,000	600,000	6,000,000
Chemicals used	Liquid chlorine	Bleach—plans for other chemicals (1915)	Sulfate of alumina	None	Sulfate of alumina	Bleach—sulfate of alumina occasionally to assist sedimentation	Liquid chlorine

* In addition, filtered water reservoir, $417,000; pumps, $90,000, boilers, $67,000; misc machinery, $218,000; pumping station and connections, $345,000; intakes, etc, $213,000. (Bids in 1905 for plant as originally built)

Table 297. Data Relating to Typical Slow Sand Filter Plants.—(*Continued*)

	Philadelphia,[1] Pa.	Pittsburgh, Pa	Washington, D. C	Toronto, Ont.	Springfield, Mass.	Albany, N. Y.	Wilmington, Del.
Manner of application of chemicals.	Added to filter effluent	Filter effluent by hand	Applied to aqueduct below Dalecarlia resr. — Georgetown resr. forms settling basin	. . . .	Applied when necessary to tunnel leading to sedimentation basin	Applied to final effluent through orifices by gravity	Applied to final effluent
No. of filter units	65[2]	56	29	12	6	8[7]	6[10]
Net area of filter surface (acres).	48.75[2]	56	29	9 6	3	5.6[7]	2[10]
Depths of filtering materials (inches)	24 filters (36 in) 41 filters (28–30 in)	Gravel (12) Sand (24) to (48)	Gravel (12) Sand (40)	Gravel (12) Sand (42)	Gravel (12) Sand (36)	Gravel (12) Sand (48)	Gravel (14) Sand (24)
Sizes of filtering material (millimeters).	Final filters (0 26–0 39), U. C. = 2.6 Pre-filters (30 in gravel, 12 in. sand, 0.8–1 0.)	Eff size 0 29 Uniformity coef 2.1	Eff. size 0 32 Uniformity coef 1 77	Eff size (0.25–0 35) Uniformity coef 3 00	Eff size 0.25–0 35) Uniformity coef 3 00	Eff. size 0 31 Uniformity coef 2.3	Eff. size 0.23 Uniformity coef. 1.83
Cleaning of filter system.	Nichols machine—dirty sand scraped by hand and forced by ejectors to machine where it is washed and returned to place without removal from filter	Scraped by hand, ejected to washers and placed in idle bed	Scraped by hand, ejected to washers and returned to idle bed or stored in elevated bins	Scraped by hand, ejected to court, washed and stored. Returned by ejectors	Scraped by hand, removed by ejectors and stored in court Returned by ejectors	Scraped by hand, removed by ejectors and stored in court. Returned by ejectors[8]	Scraped and cleaned without removal from filter, by means of "Blaisdell Machine"
No. of coagulating or subsiding basins.	Under construction (1915)	3	3	None	1	1	1
Control of rate of filtration.	By circular, floating weir, whose depth below surface is fixed by a float which may be regulated from floor above	Simplex controller and by hand	Hand regulation—effluent flows through Venturi meters provided with indicators to show rate	Hand regulation—Venturi meters and indicators	Hand regulation—Venturi meters and indicators	Hand regulation—effluent flows through an orifice on which a constant head is maintained	Hand regulation—Venturi meters and indicators

[1] Torresdale only; other filters are· Queens Lane, 16 1 acres; Belmont, 13 53 acres; Upper Roxborough, 5 60 acres; Lower Roxborough, 2 65 acres.
[2] Final filters only. Pre-filters have area of 145,800 sq ft. [3] Not including land, administration or experimental station.
[4] Includes system of contact and A-frame baffles for removing colloidal matter. See p. 732.
[5] Not including preliminary treatment plant added in 1910 at cost of $23,000. [6] Not including land, eng. or overhead charges
[7] Sixteen pre-filters also used—built in 1907–08. Total area, 13,000 sq. ft. [8] Nichols machine also used. [9] Without pre-filters.
[10] Also 10 pre-filters having area of 14,500 sq. ft.

Table 298. Data Relating to Typical Mechanical Filter Plants

	St Louis, Mo.	Cincinnati, Ohio	New Orleans, La.	Minneapolis, Minn.	Montreal, P Q
Plant put in operation	1915	1907	1909	1913	1912
Designer	Ed E. Wall	G H Benzenberg	Geo. G. Earl.	Hering & Fuller, Jones	F. H Pitcher
Population (1910)	687,000	363,600	339,100	301,400††	280,000‖‖
Total cost	$1,357,000* (filters and appurtenances)	Coagulating basins $304,913 Filters 734,102‡ Filtered water basin 121,362 Pipe lines 29,701 Cleaning, grading, etc. 33,360 $1,223,438	$2,515,000§	$960,000‡‡	$400,000 including land and low-lift pumping station.
Cost per million gals. capacity	$8480 (filters only)	$10,900	±$28,000	$18,460	$13,800
Source of supply	Mississippi river	Ohio river	Mississippi river	Mississippi river	St. Lawrence river°
Rated capacity (gals per day)	160,000,000	112,000,000	50,000,000	52,000,000	30,000,000
Total capacity of subsiding or coagulating basins (gals)	237,000,000	22,973,000	30,000,000‖	5,600,000§§	1,120,000
Total capacity of filtered water basin (gallons)	20,000,000 (Baden) 60,000,000 (Bissels Pt) 58,000,000 (Compton)	19,000,000	15,000,000	45,000,000	600,000
Chemicals used.	Lime Sulfate of iron Sulfate of alumina Liquid chlorine	Lime Sulfate of iron	Lime Sulfate of iron	Lime Sulfate of alumina Bleach	Sulfate of alumina Bleach
Manner of application of chemicals	Lime added in mixing channel, then sulfate of iron, water settled, and sulfate of alumina added. Chlorine added to effluent	Sulfate of iron introducted in "Circulating Chamber." Lime solution on way to coagulation basin	Lime added in mixing reservoirs and sulfate of iron as water passes to coagulating basin	Lime added in mixing channel and sulfate of alumina further along channel. Bleach at clear water basin	Chemicals added by gravity through constant head orifice
Number of filter units	40	28	10	16	15
Net area of filter surface (sq. ft)	56,000	39,200	14,300	17,800	10,440
Depths of filtering materials (inches)	Gravel (12) Sand (30)	Gravel (14) Sand (30)	Sand (36)	Gravel (14) Sand (30)	Gravel (9) Sand (27)
Sizes of filtering material (millimeters)	Eff size 0.5–0 6 Uniformity coef. 1 65	Eff. size 0 39 Uniformity coef 1 35	Eff size 0 32–0 36 Uniformity coef 1.65	Eff size 0 35–0 44 Uniformity coef 1 65	Eff size 0.4–0 6 Uniformity coef. 1.5–1.6
Cleaning of filter system	Reverse flow of water Velocity 2.0 ft per min.	Reverse flow of water Velocity 17 ins per min.	Reverse flow of water Velocity 25 ins. per min	Reverse flow of water Velocity 19 ins. per min.	Reverse flow of water with agitation by compressed air
Number of subsiding or coagulating basins.	9†	3	4**	4	2
Control of rate of filtration	Controllers of the Venturi meter type	Balanced valve and orifice controlled by difference in pressure on opposite sides of orifice	Balanced valve controlled by head on orifice and system of balanced floating cylinders	Simplex rate controller	Rate controller of Venturi type

* Includes engineering but not real estate. The appraised value of old work incorporated in the new plant is as follows: Land, $52,000. Settling basin, 1,923,000. Coagulant house, 160,000.
† Besides grit chamber. ‡ Not including engineering, legal or other overhead charges, pipe line to reservoirs, or real estate. § Includes cost of pumping station, cost of preliminary investigations, wharf and intake, land, and engineering ‖ Not including two grit chambers, total cap. 6,600,000 gals.
** Also 2 mixing chambers, 2.5 mg. each. †† 350,000 (1915). ‡‡ Includes $135,000 for covering and improving filtered water basin. §§ Also settling reservoir, cap. 75,000,000. ‖‖ Supplied by Montreal Water and Power Co. City also has double slow sand filter, not yet completed (1915). °Mixed with Ottawa river.

Table 298. Data Relating to Typical Mechanical Filter Plants.—(*Continued*)

	Little Falls, N. J	Louisville, Ky.	Harrisburg, Pa	Evansville, Ind	Belfast, Me.
Plant put in operation	1902 (*a*)	1909	1905	1912	1914
Designer	G W Fuller	Chas Hermany	J. H. Fuertes	Alvord & Burdick	R. S. Weston
Population (1910)	250,000 (pop. supplied approx)	223,900	64,200	69,600	4,620
Total cost	$600,000 with additions	Filters, clear water basin & laboratory $1,266,000 Coagulating basin $130,000	Land, intake, grading low lift pumping sta and pipe across river $165,683 Sed and coagulation basins 33,065 Clear water basin 15,953 Filters (including superstructure) 80,527 Total $295,928 §	$246,000, including low-lift pumps and exterior piping.	$16,000 (inc. changes in pumping station
Cost per million gals. capacity	$18,750	$17,800	$10,800 (filters and basins) $24,600 (total)	20,500 ‖	$16,000
Source of supply	Passaic river	Ohio river	Susquehanna river	Ohio river	Little river
Rated capacity (gals. per day)	32,000,000	78,000,000	12,000,000	12,000,000	1,000,000
Total capacity of subsiding or coagulating basins (gals)	7,000,000	12,000,000 *	334,000 ‡	2,500,000	84,000
Total capacity of filtered water basin (gallons)	3,500,000	25,000,000	625,000	1,900,000	31,000
Chemicals used	Sulfate of alumina Soda ash occasionally Chlorine	Sulfate of alumina Bleach	Sulfate of alumina Soda ash when necessary Bleach	Sulfate of iron Lime Bleach	Sulfate of alumina Soda ash when necessary
Manner of application of chemicals	Added to coagulant basin through manifold at mouth of main inlet pipe, chlorine to effluent	Chemical added to coagulant basin through constant head orifice	Chemicals added at entrance to coagulating basin	Chemicals added to baffled forebay near entrance to settling tanks	Chemicals added near entrance to coagulating basin
Number of filter units	32	18	12	12	2
Net area of filter surface (sq ft)	11,520	26,000	5,200	4,416	384
Depths of filtering materials (inches)	Gravel (12) Sand (30)	Sand (26) Gravel (14)	Gravel (7) Sand (30)	Gravel (9½) Sand (30)	Gravel (6) Sand (30)
Sizes of filtering material (millimeters)	Eff size 0 44–0 46 Uniformity coef. 1 50	Eff size 0 47 Uniformity coef. 1 3	Eff size 0 38 Uniformity coef 1 3	Eff size 0.36–0.55 Uniformity coef 1 3	Eff size 0.41 Uniformity coef. 1 58
Cleaning of filter system	Reverse flow of water with agitation by compressed air	Reverse flow of water Velocity 2′ per minute	Reverse flow of water with agitation by compressed air	Reverse flow of water with agitation by compressed air; velocity of wash water 11″ to 14½″ per minute	Reverse flow of water Velocity = 18 ins per min.
Number of subsiding or coagulating basins	1	2†	2	3	1
Control of rate of filtration	Weston type of moving orifice operated by float	Telescopic tube and float maintain constant head on orifice	Float actuates a valve so that constant head on orifice is maintained	"Pressure Disk" type of rate controller	Rate controller of Venturi type

[(*a*) Additions made in 1914; basins designed for 64 mg] * Also settling basin of 100,000,000 gals capacity. † 1, 8 mg ; 1, 4 mg. ‡ Also settling reservoir holding 4,000,000 gals. § Includes engineering but not preliminary investigation ‖ High cost due to expensive foundations and special construction to meet fluctuating river.

TYPICAL DATA ON COST OF WATER PURIFICATION PLANTS

General Data (Approximate)

Estimated normal cost of covered slow filters per acre with all appurtenances including small pump well,*	$100,000
Cost of similar filters uncovered, per acre,	75,000

On the above basis the estimated costs of *slow filters* per million gallons are as follows:

Type	Rate of filtration	Cost per mgd.†
Covered	5 mgad.	$20,000
Covered.............	3 mgad.	33,333
Open...............	4 mgad.	18,750
Open................	3 mgad.	25,000

Cost of *rapid filters* with coagulating basins and small pump well providing storage of wash-water and a small reserve for pumps may be estimated to be $14,000 per mgd. complete. This cost implies concrete construction. Wooden tanks are cheaper. Plants may cost as low as $8000 per mgd. Cost of open coagulating basins is estimated to be $10,000 per mg. storage. Cost of covered basins $15,000 mg.

Table 299. Usual Cost of Chemicals Used as Coagulants, Etc.

Kind	Cost per ton	Cost per lb.
Sulfate of alumina.........................	$20	$0 01
Ferrous sulfate, crystallized.....................	12	0 006
Lime, 90 per cent, CaO	8	0.004
Soda, 95 per cent. Na_2CO_3............	30	0 015

Cost of Low-lift Pumping‡ (see also page 462)

Cost of low-lift pumping is estimated to vary between $0.02 and $0.20 and to average $0.15 for 1 mg. 1 ft. high.

Average additional lift for mechanical filters.....	15 ft.
Average additional lift for slow filters........................	10 ft.
Cost of low lift pumping, mechanical filters	$2.25 per mg.
Cost of low lift pumping, slow filters	1 50 per mg.

Cost of Wash-water

Cost of filtered wash-water is assumed to be	$8 00 per mg.
Cost of pumping wash-water 25 ft. @ $0.15 per mg., 1 ft. high ..	3 75
	$11.75 per mg.

Average quantity of filtered wash-water, well-designed plants, 2.5 per cent.
Equivalent to about $0.30 per mg. filtered.
Two per cent. wash-water would cost $0.24 per mg.
Three per cent. wash-water would cost $0.36 per mg.
Four per cent. wash-water would cost $0.48 per mg., etc.

* By small pump well is meant one large enough to compensate for differences between rates of filtration and pumping to distribution systems, respectively.

† Cost does not vary strictly as rate on account of the large proportion of reserve required for higher rates.

‡ If filters are placed near high-level reservoir, so that pumping is an addition to high lift, cost will be much less, say $0.02 to $0.06 per mg. 1 ft. high for large quantities.

SLOW FILTER PLANTS

Cost of Slow Filters and Appurtenances

Cost of covered slow sand filters including small pump well, per mg. filtered for different rates and different numbers of bacteria per c.c. in effluent.

Rate of filtration mgad	Average maximum number of bacteria per c c. in effluent	Cost per mgd.
10.0	200	$10,000
6 0	100	16,667
5 0	75	20,000
4 0	60	25,000
3 0	50	33,333
2 5	40	40,000

Cost of Modified Slow Sand Plants Complete

Cost of modified slow sand filters with coagulating basins or roughing filters for color and turbidity removal requiring 24 hrs. storage in basins before filtration for different degrees of color and turbidity, is estimated as follows:

Rate of filtration mgad.	Color, raw water	Turbidity, raw water	Cost per mgd
10 0	0 upward	0–100 200 upward	$25,000 27,800

RAPID FILTER PLANTS

Average fixed charges from figures given in several filter-plant reports and where not given, estimated at 6 per cent. = $4.67 mg.

Approximate ratio of rated capacity of small rapid plants to water actually delivered to mains is 3 to 2.

Estimated fixed charges on filter plant (first cost $14,000 per mgd.) at 6 per cent. when operating at full capacity, $2.33 mg. At two-thirds capacity, $3.49.

The above costs are higher than those of the largest and best plants constructed under favorable conditions, but lower than those of plants where unusual conditions prevailed.

Cost of Rapid Filter Plants, Complete

Cost of construction of rapid filter plants including necessary subsiding and coagulating basins and small filtered water basin or pump well for different degrees of turbidity or color of unfiltered water.

Rate of filtration, mgad.	Turbidity of raw water	Color of raw water	Cost per mgd.
125	0–100 100–300 300 upward	0–25 25–75 75 upward	$14,000 16,500 18,500

OPERATION

The operation of a filter is fully as important as its design.

General inspection of a purification plant is constantly necessary, particularly if the character of the raw water fluctuates rapidly. In slow filters, the

condition of the sand is of utmost importance, and in large plants the management of the sand-handling processes requires a great deal of skill.

Rate of filtration may be determined and regulating devices checked by closing the filter inlet and measuring the water filtered by observing the depression of the flow line and computing the volume of water equivalent thereto.

Table 300. Equivalents of Various Measures of Rate of Filtration

Rate	Million U S. gals per acre per 24 hrs.	Million Imperial* gals. per acre per 24 hrs.	U. S. gals. per sq. ft. per hour	Imperial* gals per sq. ft. per hour	Cu. ft. per sq. yd. per hour	Vertical velocity in in. per hour	Vertical velocity in millimeters per hour	Vertical velocity in meters per 24 hrs., = cu. meters per sq. meter per 24 hrs.
1 million U.S. gals. per acre per 24 hrs..	1.0	0.833	0 96	0.80	1.15	1 53	39.	0.935
1 million Imperial gals. per acre per 24 hrs	1.200	1 0	1 15	0.96	1 38	1.84	46 8	1.122
1 U.S. gal. per sq. ft. per hr. .	1 045	0 870	1 0	0 83	1 20	1.60	40 7	0.978
1 Imperial gal. per sq. ft. per hr.	1 255	1 045	1 20	1 0	1.44	1.92	48 9	1.174
1 cu. ft. per sq. yd. per hr ...	0 869	0 724	0.83	0.69	1.00	1.33	33 9	0.813
1 linear in., vertical velocity, per hr..........	0 652	0.543	0.62	0.52	0.75	1.0	25.4	0.610
1 hundred linear millimeters, vertical velocity per hr	2 566	2.139	2 46	2.05	2 95	3 94	100.0	2.400
1 linear meter, vertical velocity per 24 hrs. = 1 cu meter per sq meter per 24 hrs ..	1 069	0.891	1.02	0.85	1 23	1 64	41 7	1 0

Filter controllers, loss-of-head gages and all other automatic devices should be frequently inspected and kept in order; otherwise inferior results will be obtained.

Curves in pipes for handling sand wear rapidly. Rubber-lined hose is best for sharp bends.

Corrosion and "Red Water Plague." Pure water dissolves iron to a limited degree. Water otherwise pure, which contains carbonic, organic or other acids, attacks metals, particularly lead and iron, readily. The presence of oxygen increases the action on iron. Soft ground waters containing carbonic acid are the most, and surface waters containing clay and alkalis, the least apt to corrode metals. Many surface waters form a protective coating of organic matter on the insides of distributing pipes, thereby preventing further corrosion. The removal of protecting organic matter by filtration increases corrosion. The use of sulfate of alumina as a coagulant increases the carbonic acid and decreases the protecting clay and organic matter. Slow filters without coagulants may increase the corrosiveness of a water by the oxidation of organic matter and the production of carbon dioxide within the filter.

Peaty waters, containing humic and other organic acids, frequently corrode lead pipes and give rise to lead poisoning (plumbism). This and other corrosive action of the water may be diminished by artificially increasing the hardness. This is best accomplished by the addition of pure quick lime in a quantity great enough to neutralize the free acid and part of the bicarbonates.

* Sometimes called British. See page 623.

Sodium carbonate or hydrate may also be used, but at a greater cost. Corrosion is most apt to occur in hot water plumbing systems, and is very frequently due to the poor quality of the pipe used. When metals are not homogeneous, electrolysis is set up and the solvent action of the water is increased.

Statistics of Operation. The Committee on Statistics of Water Purification (Jour. N. E. W. W. Ass'n, Jan. 13, 1915) has recommended standard forms for filter-plant statistics. Analytical methods adopted by Am. Public Health Ass'n are recommended. See p. 788. Elimination of unnecessary analytical work is important. Determination of nitrogen in the usual form of albuminoid ammonia, nitrites and nitrates serves no useful purpose, except in special cases. Some of the simpler physical tests—turbidity, color, odor, microscopical examinations, alkalinity, iron and carbonic acid are most valuable. In special cases, dissolved oxygen and manganese should be determined. Bacteriological tests are always important.

Many records relating to engineering matters are useful and should be kept. It is not necessary to include all in published reports. Tables on pages 774 to 779, include only data of sufficient value to be published.

Sampling is as important as are analytical methods. Bodies of water are not homogenous. *Frequency of sampling* should depend upon frequency of change in character of water examined. This has a limitation which is controlled by practical and financial considerations. Results based on infrequent samples, however, are less valuable than those based upon frequent ones. Irregular sampling gives the most unreliable results.

Grade of Supervision. Control of purification plants may be graded as follows: First-grade plants are those where analyses of filtered water are made one or more times a day, and where engineering and such other data regarding operation of plant as are necessary are collected by one or more attendants constantly employed. Second-grade plants are those where analyses are made regularly, say once a week or once a month by a trained analyst, and where an attendant constantly on duty makes simple daily tests. Third-grade plants are those where analyses are made irregularly and infrequently, and where no daily tests are made by the attendant.

Standard Forms for Statistics. Terms used in the various tables* pages 774 to 779 recommended by N. E. W. W. Ass'n, may be explained as follows: (See page 780.)

* Blank forms may be purchased from N. E. W. W. Ass'n, Tremont Bldg., Boston.

Table —. Chemical and Microscopic Character of (**) Water
(Parts per Million)
FOR THE YEAR ——

Month	Chlorine	Total hardness	Permanent hardness	Alkalinity			Free CO_2		Dissolved oxygen	Iron	Suspended matter	Oxygen consumed	Micro-organisms	Remarks
				Av	Max.	Min	Av.	Max.					Std. units per c c.*	
Jan. ↓ Dec														
Average														

* Use standard units; if numbers are used, so state
(**) Water to be designated by name of source, *e.g.*, "Hudson River," "Wells," "Lake George."

Table —. Turbidity and Color of ( ) Water
(Parts per Million)
FOR THE YEAR ——

Month	No of test days	Turbidity		Turbidity, variations· No. of test days							No. of test days	Mean color	Color variations No. of tests days				
		Mean	Median	0–10	11–25	26–50	51–100	101–250	251–500	Above 500			0–10	11–25	26–50	51–100	Above 100
Jan. ↓ Dec.																	
Total																	
Average																	
Per cent. time																	

Table ——. Chemical Character of Water Delivered to Mains
(Parts per Million)
For the Year ——

Month	Chlorine	Total hardness	Permanent hardness	Alkalinity		Free CO_2		Iron	Oxygen consumed
				Average	Minimum	Average	Maximum		
Jan ↓ Dec.									
Average									

Table ——. Numbers of Bacteria in Water Delivered to Mains
For the Year ——

Month	On gelatin at 20° C.									On agar at 37.5° C.								
	No. of test days	Mean per c.c.	Median per c.c.	Variations in numbers: No. of test days						No. of test days	Mean per c.c.	Median per c.c.	Range of counts				
				0–10	11–25	26–50	51–100	101–250	Above 250				0–10	11–25	26–50	51–100	Above 100
Jan. ↓ Dec.																	
Total																	
Average																	
Per cent time																	

Table ——. Bacteria in ( ) Water. (Same form as Table 234, page 662)

Table ——. Turbidity and Color of Water Delivered to Mains. (Same form as Table 239, page 665)

Table —. Numbers of Bacilli Coli in Water Delivered to Mains
For the Year ——

Month	Bacillus coli							B. coli index	Remarks
	No. of test days	One c c tests			Ten c c tests			Number per c c	
		Total No	No. +	Per cent. +	Total No.	No. +	Per cent +		
Jan. ↓ Dec.									
Total									
Average									
Per cent time									

Table —. ———— Plant, City or Town of ————
Volumes of Water and Rates of Filtration. For the Year ——

Month	Volumes of water, million gallons daily				Plant in operation		Average yield, million gallons acre daily	Average rate of filtration, million gallons acre daily†	Remarks
	Filtered			Delivered unfiltered	Number of days	Average hours per day			
	Average delivered to mains	Maximum delivered to mains	Average wash water*						
Jan. ↓ Dec.									
Total									
Average									

*Includes all water wasted and used by purification plant. †Calculated on area of filter in service.

Table ——. ———— Plant, City or Town of ————

RESULTS OF OPERATION OF MECHANICAL FILTERS. FOR THE YEAR ——

Month	Average length of runs	Average loss of head		Washing			Period of plain subsidence	Period of coagulation	Remarks
				Average time in minutes		Per cent, wash water			
	Hours	Initial	Final	Air	Water		Hours	Hours	
Jan. ↓ Dec.									
Average									

NOTES.—*Air* applied (with water or separately).
Rate . . . . . cubic feet free air per minute.
Pressure pounds per square inch.
Water applied, average vertical velocity of wash water above filter surface ( ) inches per minute.
Pressure . . . pounds per square inch

Table ——. ———— Plant, City or Town of ————

RESULTS OF OPERATION OF SLOW SAND FILTERS. FOR THE YEAR ——

Month	Million gallons filtered	Number of filters ending runs*	Area cleaned—acres			Average yield of runs million gallons per acre				Average length of cycle, days	Number of filters, scraped	Total sand scraped	
			Raking	——— Method	Scraping	After first raking	After second raking	After ——— method	After scraping			Cubic yards	Depth, inches
Jan. ↓ Dec													
Total													
Average													

* A run terminates whenever a filter is stopped to reduce the loss of head.

Table ——. Numbers of Bacilli Coli in Water Delivered to Mains

FOR THE YEAR ——

Month	Bacillus coli							B coli index	Remarks
	No. of test days	One c c tests			Ten c c tests			Number per c c	
		Total No	No. +	Per cent. +	Total No.	No. +	Per cent +		
Jan. ↓ Dec.									
Total									
Average									
Per cent time									

Table ——. ———— Plant, City or Town of ————

VOLUMES OF WATER AND RATES OF FILTRATION. FOR THE YEAR ——

Month	Volumes of water, million gallons daily				Plant in operation		Average yield, million gallons acre daily	Average rate of filtration, million gallons acre daily†	Remarks
	Filtered			Delivered unfiltered	Number of days	Average hours per day			
	Average delivered to mains	Maximum delivered to mains	Average wash water*						
Jan. ↓ Dec.									
Total									
Average									

* Includes all water wasted and used by purification plant.

† Calculated on area of filter in service

Table ——. ———— Plant, City or Town of ————

RESULTS OF OPERATION OF MECHANICAL FILTERS. FOR THE YEAR ——

Month	Average length of runs	Average loss of head		Washing			Period of plain subsidence	Period of coagulation	Remarks
				Average time in minutes		Per cent, wash water			
	Hours	Initial	Final	Air	Water		Hours	Hours	
Jan. ↓ Dec.									
Average									

NOTES.—*Air* applied (with water or separately).
Rate . . . .cubic feet free air per minute.
Pressure . pounds per square inch.
Water applied, average vertical velocity of wash water above filter surface ( ) inches per minute
Pressure ....pounds per square inch.

Table ——. ———— Plant, City or Town of ————

RESULTS OF OPERATION OF SLOW SAND FILTERS. FOR THE YEAR ——

Month	Million gallons filtered	Number of filters ending runs*	Area cleaned—acres			Average yield of runs million gallons per acre				Average length of cycle, days	Number of filters, scraped	Total sand scraped	
			Raking	—— Method	Scraping	After first raking	After second raking	After —— method	After scraping			Cubic yards	Depth, inches
Jan ↓ Dec													
Total													
Average													

* A run terminates whenever a filter is stopped to reduce the loss of head.

Table ——. ———— Plant, City or Town of ————.—*(Continued from p. 777)*

Month	Number of filters to which sand is restored	Total sand restored		Water used to clean filters, gallons	Remarks
		Cubic yards	Depth, inches		
Jan. ↓ Dec.					
Total					
Average					

Table ——. ———— Plant, City or Town of ————

CHEMICALS APPLIED TO WATER. FOR THE YEAR ——

Month	Sulphate of alumina		*		*		Chlorine		Remarks
	Pounds per million gallons	Parts per million	Pounds per million gallons	Parts per million	Pounds per million gallons	Parts per million	Pounds per million gallons	Parts per million	
Jan ↓ Dec									
Average									

* Blank space to be used for other chemicals

Sulfate of alumina used contains ... % Al_2O_3 (alumina)
Ferrous sulfate used contains ...... % Fe (iron).
Bleaching powder used contains.... % Cl (chlorine)

Soda used contains % Na_2CO_3 (sodium carbonate).
Lime used contains .. % CaO (calcium oxide).

Table ——. ———— Plant, City or Town of ————

ANNUAL COST OF OPERATION AND FIXED CHARGES PER MILLION GALLONS DELIVERED TO MAINS. FOR THE YEAR ——

	Pumping* to plant	Chemicals† and treatment	Supt. and laboratory	Filter attendance	Sand handling	Wash water	Buildings and grounds	Heating and lighting	Repairs	Miscel-laneous	Total
Labor											
Materials											
Total											

* Or additional cost of pumping water because of purification † To be subdivided if more than one chemical be used.

TOTAL COST OF PURIFICATION

Total cost of operation per million gallons. $..........

Total fixed charges per million gallons $..........

Total cost of purified water per million gallons. $..........

Total cost of purified water per annum $............

Test day is period of 24 hrs. during which tests are made. This seems to be the shortest period practical to use as a statistical unit in determining efficiency of purification plants. When more than one observation is made during a test day, results should be combined by averaging to obtain single numbers representative of that day, and the fact stated. Assuming the precision of results of daily sampling to be unity, the precision of results of weekly sampling would be only 0.38; of monthly sampling, 0.18; and of yearly sampling, 0.05; while the precision of hourly sampling would be 4.90.

Period of service is the length of time during which the filter is delivering water. *Period of washing* is the length of time required to stop, wash and start the filter. *Period of operation* is the total length of time during which the filter is either filtering water or being washed.

Monthly and Annual Summaries. For the purpose of uniform tabulation, calendar months and calendar years should be used as the basis of grouping daily results.

Averages. Each monthly average, or mean, should be obtained by dividing the sum of the daily results by the number of test days. The annual average should be the mean of the twelve monthly averages.

Median. Occurrence of occasional observations which are abnormally high often causes the average to be non-representative of the month or year. For this reason the median as well as the mean should be computed for each month and year. Literally, the median is the result which lies in the middle of a series of results arranged in order from lowest to highest. If there be an odd number of tests, select the middle term; if an even number, take the average of the two middle terms. To find the yearly median, arrange the daily results for the entire year in order of magnitude, regardless of the month in which they occur. Do not take the average of the monthly median, or even the median of the monthly medians.

Per Cent. of Time. Several tables provide a horizontal line for the per cent. of time during which the results fall within the stated groups. When lines are drawn through certain spaces in this and other summaries, they indicate that the spaces should not be filled out.

Precision of Analytical Methods. For uniformity the Standard Methods of the Am. Public Health Assoc. should be adopted so far as practicable. In

Table 301. Rules for Significant Figures

Determination	Results to be reported
Numbers of bacteria.	Ordinarily use two significant figures; never more than three
Percentage of time.	To first decimal place
Percentage of B. coli tests	To nearest whole number
Chlorine	To first decimal place
Hardness	To nearest whole number
Alkalinity	To nearest whole number
Carbon dioxide	To nearest whole number
Iron	To second decimal place
Oxygen consumed	To first decimal place
Manganese	To second decimal place

general, averages of the above results should be carried out one decimal place beyond those mentioned, but in case the average is that of only a few items this additional place is neither necessary nor desirable.

B. Coli Tests. When possible use "Positive," "Negative," or "Pos." or "Neg." or (+) and (−); when no test is made, fill the space with dots (. . . .), not a dash, which might be taken for a minus sign. A zero might be taken for either "negative" or "no test." B. coli index is approximate number of B. coli per c.c., as determined from qualitative tests upon different quantities of water. For any individual sample, it may be taken as the reciprocal of the smallest volume of water used in the test which gave a positive result. B. coli index for a single sample is not very accurate. The index becomes more and more precise as the square root of the number of tests becomes larger. B. coli index may be computed from a series of results as follows: Write down in order of magnitude of quantities of water examined the percentages of positive tests, expressed as decimals of 100. Take the differences between these percentages. Multiply each difference by the reciprocal of the quantity corresponding to the larger of the two percentages from which such difference was taken. The sum of these products will be the B. coli index.

Efficiency of Operation. The common method of judging efficiency by percentage removal of bacteria is unsatisfactory for several reasons:

(1) Bacteria found in effluent are not all from the raw water. A certain and varying number of them represent growths in the lower part of the filter. Bacteria from this source do not vary in number with the number of bacteria in the raw water, but with the rate of filtration, with disturbances occasioned by the collection of air within the filter, and with other factors of operation.

(2) It requires time for water to pass from the point where the raw sample is taken to the point where the filtered-water sample is taken, sometimes several hours, and in the case of a water which changes rapidly it is necessary to make an allowance if a correct comparison is to be obtained.

(3) Percentage removal is to a certain extent a function of the number of bacteria in the raw water. The percentage removal is relatively high when the raw water contains large numbers of bacteria and relatively low when the raw water contains few bacteria. One reason is that the bacteria which develop within the filter are a smaller proportion of the total number in the effluent when the number passing the filter is large.

(4) Percentage removal does not necessarily vary with the number of bacteria left in the filtered water.

(5) Infectiousness does not necessarily vary with the number of bacteria but in most waters there will doubtless be some connection between the two. A water polluted with surface wash would doubtless increase in infectiousness and the numbers of bacteria would increase with increased stream flow. A water regularly polluted with sewage, other things being equal, would increase in infectiousness as stream flow decreased, that is, as dilution became less, while the numbers of bacteria might decrease. Hence, there seems to be no very definite relation between infectiousness and percentage removal, which percentage varies according to the numbers of bacteria in the raw water.

(6) It would be better to use "percentage of bacteria remaining;" the num-

bers show to better advantage variations in operation of the filter, and the figures are smaller and easier to handle; for example, when two filters, with the same raw water, are operating so as to produce 98 and 99 per cent. "removals," the numbers of bacteria remaining in the effluent of the first will be twice as many as in the second. Comparing one filter with an efficiency of 99 per cent. with another of 99.9 per cent., both operating with the same raw water, the effluent of the first contains 10 times as many bacteria as the second.

(7) A single figure showing percentage removal during a certain time gives no idea of the regularity of operation.

(8) A common fault of specifications is the provision that the plant shall show a certain percentage removal when the numbers of bacteria in the raw water are below a certain limit. This specification is inadequate. A good purification plant is one which produces a satisfactory effluent every day and every hour of the day; efficiency should be indicated by the percentage of time during which the plant produced a good effluent.

RESULTS OF OPERATION

Table 302. Cost of Water Purification in Typical Cases

These data based on a plant of 5 mgd. capacity operating at a $\frac{2}{3}$ rate.

A. Turbidity Removal

Typical cost of water purification by rapid purification plant with subsiding and coagulating basin and appurtenances, when average turbidity = 250 p.p.m.

Cost of construction	$18,500.00 per mgd.
Interest and sinking fund (6 per cent.)	$3.04 per mg.
Interest and sinking fund when operating at $\frac{2}{3}$ rated capacity	$4.56 per mg.

Typical cost of filtration, $\frac{2}{3}$ rated capacity

		$ mg.
Fixed charges		$4.56
Cost of operation, with sulfate of alumina.		
Pumping, 15 ft. lift	$2.25	
35 p.p.m. sulfate of alumina	2.93	
Superintendence, chemist, two attendants	1.81	
Wash water, 2.5 per cent	0.30	
Repairs and replacements (machinery) 10 per cent. on $2000 per annum	0.17	
Other repairs 0.5 per cent. on $16,500 per annum	0.07	
Light, heat, etc	0.20	
Total operation		7.73
Ditto with ferrous sulfate (copperas) and lime.		
Cost of chemicals		
30 p.p.m. ferrous sulfate	1.51	
15 p.p.m. lime	0.37	
Extra superintendence	0.41	
	2.29	
Saving over sulfate of alumina	0.64	
Total		7.09
Total operation		$7.09 to $7.73
Total cost of purification per mg		$11.65 to $12.29

B. Color Removal

Same conditions except for the removal of 100 p.p.m. color instead of turbidity.

	$ mg.
Cost of sulfate of alumina, 40 p.p.m.	$ 3.35
Fixed charges and other operating costs	9.36
Total cost	$12.71 mg.

In many places cost of chemicals for color removal and fixed charges (because of reduced basin size) could be greatly reduced by use of chlorine—cost $0.10 mg. Probably would reduce basin cost by $2000 mgd. and cost of chemicals $1.50 mg., making filtered water cost $10.82 mg.

C. Bacteria Removal

Typical cost of purification by slow sand filters treating water whether from subsiding basins or from natural sources containing less than 30 p.p.m. turbidity and less than 20 p.p.m. color; covered filters; rate of filtration 5 mgad. gross; 4 mgad. net; capacity 5 mgd.; consumption 4 mgd.

Cost of construction	$20,000 0
Fixed charges @ 6 per cent. 5 mgad.	3.29 mg.
Fixed charges 4 mgad.	4.12 mg.

Cost of filtered water per mg.

Fixed charges		4.12 mg.
Cost of operation:		
Pumping, 10 ft. lift	$1.50	
Scraping and washing sand with ejectors, including labor	0 75	
Superintendence, 1 chemist and 1 attendant	0 82	
Repairs, 0.5 per cent. on $20,000 per annum	0 05	
Light, heat, etc.	0 10	
Total cost of operation		3 22
Total cost of purification per mg		$7 34

Low cost for slightly turbid and colored waters should not be compared side by side with costs of purifying by the rapid system with coagulant without correcting for basin sizes and amounts of chemical. Data for rapid filters (typical case) are based on water having a turbidity of 200 to 300 and a color of 100 p p.m.

John Gant (J. Am. W. W. Ass'n, 2, 596, *et seq.*, 1915) gives costs of sand handling and cleaning slow filters as follows:

Table 303. Cost of Sand Handling

City	Scraping			Raking		
	Per cu. yd	Per mg.	Per acre	Per cu yd.	Per mg.	Per acre
Albany, 1899–1900	.	$0 25			.	.
Albany, 1912–1914	.	$0 07			$0 01	.
Pittsburgh, 1911–1914	$0.175	$0 314	$25 42	$0 061	$0 064	$8 90
Springfield, 1914		0 36			0 09	..
Washington, 1914.	$0.096	$0.070	$16 08	$0.042	$0 03	$6 64

City	Ejecting, washing and transporting			Restoring		Method
	Per cu. yd	Per mg.	Per acre	Per cu. yd.	Per mg	
Albany, 1899–1900	...	$1 09		..	$0.39	Hand and ejector.
Albany, 1912–1914	...	$0.28	.	.....	..	Nichols machine.
Pittsburgh, 1911–1914.	$0 374	$0.663	.	$0 087	$0 147	Hydraulic.
Springfield, 1914.	..	$0 17	..	..		Hydraulic.
Washington, 1914	$0.168	$0 110	$28.32	$0.08	$0.06	Hydraulic.

Costs at other plants are as follows:

Toronto; portable washer; wash water = 0.33 per cent.; total cost of cleaning filters, handling sand, including repairs = $0.61 cu. yd.

Denver; ejectors, washers, restored by hand; total cost of cleaning filters and washing sand = $0.73 cu. yd.

New Haven, Conn.; hand and hydraulic; cost = about $0.43 cu. yd.

Lawrence, Mass.; ejectors, washers, hand restoring; cost = about $0.25 cu. yd.

Philadelphia; various; summary of costs as follows:

Method	Cost	
	Per cu. yd.	Per mg.
Removal by hand	$0.43	$1 46
Piling	...	0 58
Brooklyn	...	1 68
Restoring sand,	0 26	
Nichols machine	0 41	1 22
Raking	....	0 12
Spading		0 38

Wilmington, Del.; Blaisdell machine, machine idle 70 per cent. of available working time; average power per bed 115 kw.; cost of power per bed = $3.45; cost of cleaning = $1.23 per mg. (approximate).

Middleboro, Mass.; Deferrization plant; hand scraping and raking, no washing. Cost of raking = $0.03 per mg. delivered to mains. Total cost of cleaning filters, including water used for flushing preliminary coke trickler = $1.08 per mg. delivered to mains.

ÆSTHETIC CONSIDERATIONS

The total per capita cost of water usually ranges between $2.00 and $4.00 per annum, while the cost of purification usually ranges between 10 cents and 30 cents per capita per annum. Consequently the additional cost of making a supply agreeable as well as safe is inconsiderable. Consumers supplied with water of bad appearance will resort to spring water of unknown history, and thereby incur more expense than if they paid their share of the cost of proper purification at a central plant. In addition, there is the danger of drinking polluted spring water.

TYPHOID FEVER

Table 304. Annual Typhoid Fever Death Rates for Various American Cities in Deaths per 100,000 Population

(Geo. W. Fuller)

Year	Chicago	Milwaukee	Detroit	Cleveland	Buffalo	Toronto	Boston	New York	Manhattan and the Bronx	Philadelphia	Baltimore	Washington	Cincinnati	Pittsburgh	San Francisco	St. Louis	New Orleans
1880	34	37	....	44	. . .	44	42	25	---	58	59	54	70	135	34	40	24
1881	105	47	...	99	67	66	56	38	---	74	58	46	71	151	37	53	30
1882	82	32	....	68	69	71	57	32	---	73	47	66	55	158	61	45	33
1883	62	25	....	65	34	81	52	32	---	64	35	62	51	107	64	41	23
1884	56	31	....	60	42	63	56	29	---	71	42	76	56	70	58	42	25
1885	75	28	....	34	23	59	40	27	---	64	42	65	42	76	44	31	17
1886	69	30	39	56	28	46	34	26	---	64	39	71	54	68	49	30	13
1887	50	31	65	52	34	61	44	26	---	63	41	78	142	128	23	28	15
1888	47	42	46	47	29	52	40	23	---	77	42	87	70	87	33	31	19
1889	48	28	31	74	30	46	43	24	---	71	47	94	49	95	55	33	17
1890	92	41	19	69	40	93	35	22	---	64	59	112	69	132	44	31	21
1891	174	33	34	50	51	100	34	23	---	64	35	82	62	101	42	29	24
1892	124	30	94	59	34	47	29	22	---	40	44	89	40	100	32	93	20
1893	54§	35	42	52	37	45	31	21	---	41	49	86	44	111	32	44	15
1894	38	25	28	29	59	26	29	17	---	33	47	91	55	56	37	34	29
1895	38	26	24	35	26	27	33	17	---	40	36	86	39	78	34	21	44
1896	53	18	23	43	20	27	31	16	---	34	38	58	52	62	26	20	33
1897	29	12	15	23	20	18	33	16	---	33	37	48	32	64	19	23	52
1898	41	17	22	34	27	16	34	21	---	52	36	71	33	73	41	17	66
1899	27	17	13	32	24	20	30	16	---	75	29	73	37	111	22	23	55
1900	20‡	17	27	54	26	20	26	21	---	35	35	78	39	144	13	29	40
1901	29	22	21	36	27	17	25	20	---	34	27	60	55	125	22	34	48
1902	45	15	24	33	33	13	24	21	---	44	42	78	62	134	27	37	45
1903	32	17	20	114	35	16	20	17	---	69	36	48	42	134	.	47	39
1904	20	14	20	48	23	23	23	17	---	53	37	47	79	142	27	36	36
1905	17	23	21	15§	23	16	20	16	---	48	36	48	40	99	24	20‖	32
1906	19	31	24	20	23	26	20	15	---	72	35	50*	70	130	. ..	17	30
1907	18	26	35	19	28	22	10	17	---	59	41	35	45	125	....	15	55
1908	16	17	22	13	20	20	25	12	11	35	33	37	18*	45	....	14	33
1909	13	23	23	13	23	23	14	12	12	21*†	25	33	13	23*	.. .	16	29*
1910	14	45	20	19	20	41	12	12	11	17	42	23	6	28†	.	13	32
1911	11	19	17	15	25§	20*	9	11	10	14	27	21	11	26		11	31
1912	8	25	18	6†	11	12	8	10	7	13	24	22	7	13	14	14-	14
1913	11	11	29	13	15	10	8	7	6	16	26	.	6	19	. ..	16	17
1914	7	8	12	8	16	7	9	6	5	7	22		6	...		11	20

Note.—New York included Boroughs of Manhattan and the Bronx through 1898—since 1898 Greater New York has included also the Boroughs of Brooklyn, Queens and Richmond.

* Filtration effective for city. † Sterilization of water supply begun. ‡ Drainage Canal in service. § New intake. ‖ Coagulation introduced.

Reduction in typhoid fever following filtration in northern cities like Pittsburgh and Cincinnati has been remarkable; similar reductions have taken place in northern cities where the water has been disinfected. Cities farther south, like Washington, Knoxville and New Orleans, show no such marked reduction in the typhoid death rate following improvements in water supply. There has been no marked difference between the typhoid fever death rates of Washington and Richmond. It is believed that what improvement has been made in these cities has been due to other causes than water supply.

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CHAPTER XXXVIII

EXAMINATION OF WATER

Government Examination of Drinking Water on Railroad Trains. In Bulletin No. 100 of Hygienic Laboratory of U. S. Public Health Service, Treasury Dept., Richard H. Creel (Nov., 1914) pays particular attention to the "bile presumptive test for colon bacillus." He deals especially with anaerobic bacilli, found in water of train coolers, which complicate ordinary so-called coli tests. "The difference between the actual *B. coli* percentage in the 1000 samples examined and that which would have resulted had the lactose bile presumptive test been used is as follows, the comparison applying only to tubes containing water in 10 c.c. amounts:

Number of samples	Gas	B. coli	Anaerobes	Actual B. coli, per cent	Bile presumptive test, B. coli, per cent
1000	421	91	330	9 1	22 1

The margin of error is greater in relatively pure water than in water of moderate pollution.

"Estimated according to the different trains, the comparison is as follows:

.	Actual *B. coli*, per cent confirmed	Percentage of *B coli* according to lactose bile presumptive test	Number of samples
Train 301, Pennsylvania Railroad (sleeping cars).	4 7	25 0	151
Train 84, Seaboard	6 3	17 5	59
Train 305, Pennsylvania Railroad (sleeping cars).	3 4	20 0	125
Train 305, Pennsylvania Railroad (coaches)	5 3	16 0	57
Train 82, Atlantic Coast Line (coaches)	32 0	54 0	50
Train 82, Atlantic Coast Line (sleeping cars)	14 0	17 0	144

"It will be noted that there is a wide divergence between the real and the presumptive percentage when the water is fairly pure, but that in moderately polluted water, as that from Train 82, there is less discrepancy. In the latter class of water the approximation of the two figures seems to be due to the inhibiting effect of lactose bile on the *B. coli.* From the foregoing results the 'presumptive test' for *B. coli* does not seem applicable for analyzing water of moderate pollution, as its employment would often result in condemning a water of acceptable standard of purity."

Standardization of Bacterioscopic Methods of Water Examination. In the autumn of 1914 a committee of thirteen leading bacteriologists and sanitarians appointed in March, 1914, by the Royal Institute of Public Health of Great Britain, reported on this subject. "The committee do not think it is

practicable to lay down any fixed standards to govern all cases. Speaking generally, too much stress should not be laid on the number of microbes present in a water, unless the B. coli tests yield confirmatory results. A good water should not contain any B. coli in 100 c.c., but a water containing B. coli in 100 c.c. should not necessarily be objected to without the examination of further samples. Experience has shown that even initially impure waters may be purified at a reasonable cost, so as to yield no B. coli in 100 c.c. in the majority (about 75 per cent.) of samples examined. It is much more difficult to suggest a standard by which a water should be condemned. All that the committee feel justified in stating is that the further a water departs from the above standard of purity (no 'lactose, + indol, + ' B. coli in 100 c.c.) the greater is the suspicion attaching to it, unless the local conditions and circumstances are such as to exclude undesirable pollution."

Table 305. Occurrence of B. Coli in Water Supplies
(Geo. W. Fuller)

City	Source of supply	Year	Per cent positive of B coli in 1 c c.			
			Raw water	Treated water	Kind of treatment	Presumptive test
Birmingham, Ala.	Five Mile creek	1910	84 0	8 3	Filtered	Yes
Birmingham, Ala.	Five Mile creek	1911	78 0	9 7	Filtered	Yes
Birmingham, Ala.	Five Mile creek	1912	89.0	5.7	Filtered and partly sterilized	Yes
Birmingham, Ala.	Five Mile creek	1913	78.0	0.0	Filtered and partly sterilized	Yes
Lawrence, Mass.	Merrimac river	1910	100.0	7.8	Filtered	No
Lawrence, Mass.	Merrimac river	1911	100 0	8 4	Filtered	No
Lawrence, Mass.	Merrimac river	1912	100 0	16 9	Filtered	No
New York City	Croton Lake, head	1911	65 0	.....	....	Yes
New York City	Croton Lake, dam	1911	21 0	....	....	Yes
New York City	Croton Lake, head	1912	42 0	....	.....	Yes
New York City	Croton Lake, dam	1912	35 0	...	..	Yes
New York City	Croton Lake, 135th street	1911	....	29.8	Sterilization	Yes
New York City	Croton Lake	1912	....	10 0	Sterilization	Yes
Philadelphia, Pa.	Torresdale supply	1912	...	4.1	Filtered and partly sterilized	Yes
Little Falls, N. J.	Passaic river	'11–'12	89 0	6.1	Filtered and partly sterilized	Yes
Washington, D. C.	Potomac river	1910	26 9	0 0	Filtered	No
Washington, D. C.	Potomac river	1911	16 9	0 8	Filtered	No
Washington, D. C	Potomac river	1912	43 5	0 0	Filtered	No
Washington, D. C.	Potomac river	1913	45 5	.	.....	No
Harrisburg, Pa.	Susquehanna r	1910	55 0	0 17	Filtered and sterilized	Yes
Harrisburg, Pa.	Susquehanna r.	1911	77 0	0.61	Filtered and sterilized	Yes
Harrisburg, Pa.	Susquehanna r.	1912	47.0	0 81	Filtered and sterilized	Yes
Clarksburg, W. Va.	Monongahela r	'11–'12	....	0.3	Filtered and sterilized	Yes
Clarksburg, W. Va.	Monongahela r.	'12–'13	67.0	....	....	Yes
New Orleans, La.	Mississippi river	1913	39 0	3 1	Filtered	Yes
Columbus, Ohio	Scioto river	1913	69 0	0 5	Filtered and sterilized	Yes
Cincinnati, Ohio	Ohio river	1910	91.0	0.8	Filtered and sterilized	Yes
Cincinnati, Ohio	Ohio river	1911	85.0	5.2	Filtered and sterilized	Yes
Cincinnati, Ohio	Ohio river	1912	93.0	3.0	Filtered and sterilized	Yes
Cleveland, Ohio	Lake Erie	1911	11 1	0 0	Sterilized	Yes
Albany, N. Y.	Hudson river	1912	100 0	5 0	Filtered	Yes
Wilmington, Del.	Brandywine. cr.	1911	99 3	....	....	Yes
Wilmington, Del.	Brandywine cr.	1912	99 2	11 2	Filtered	Yes
Bangor, Maine	Penobscot river	1913	86.0	0.89	Filtered and sterilized	Yes
London	Thames river	1911	87 9	....	Filtered	No
London	Lee river	1911	86 7		Filtered	No
London	Chelsea stored	......	13.4	.....	Filtered	No

STANDARD METHODS

(Abridged outline of methods based upon Standard Methods for the Examination of Water and Sewage, Am. Public Health Assoc., Laboratory Section, 1913.)

Collection of Samples.* Samples should be collected under clean and aseptic conditions and examined as soon as possible. Allowable time between collection and analysis varies; reasonable maximum limits are as follows:

Physical and Chemical Analysis. Ground waters, 72 hrs.; fairly pure surface waters, 48 hrs.; polluted surface waters, 12 hrs.; sewage effluents, 6 hrs.; raw sewage, 6 hrs. Chemical samples, sterilized by addition of some germicide like chloroform (1 c.c. per liter) may, with impunity, stand a little longer before analysis. Dissolved gases should be determined *in situ*. Care should be taken to collect representative samples.

Bacteriological Examination. Samples kept at less than 10° C., 6 hrs.

Microscopical Examination. Ground waters, 72 hrs.; fairly pure surface waters, 24 hrs.; waters containing fragile organisms, immediately.

Expression of Results. Results of chemical analysis are expressed as parts per million (p.p.m.). This really means milligrams per liter. In older laboratories the results are often expressed either as grains per gal. (U. S. and Imp.), or as parts per 100,000.

Table 306. Conversion: Grains per Gallon to Parts per Million

	Grains per U S gal.	Grains per Imp. gal.	Parts per 100,000	Parts per 1,000,000
1 gr. per U. S. gal..	1.000	1 20	1.71	17.1
1 gr. per Imp. gal . .	0 835	1 00	1.43	14.3
1 part per 100,000	0 585	0 70	1.00	10.0
1 part per 1,000,000	0 058	0 07	0.10	1.0

PHYSICAL AND CHEMICAL EXAMINATION

Turbidity (due to suspended matter) is measured by comparing with a standard suspension of silica (diatomaceous earth) of such a state of fineness that a water which contains 100 mg. per liter silica is one in which the vanishing point of a 1 mm. platinum wire is 100 mm. below surface of water with eye of observer 1.2 m. above wire. Turbidity of this water is fixed at 100. Wash diatomaceous earth in HCl and water, dry, grind and pass through 200-mesh sieve. Suspend 1 grm. in 1 liter H_2O for stock solution; turbidity equals 1000. Test a portion diluted to 100 with wire, as above. Turbidity may also be determined by means of U. S. Geol. Survey turbidity rod of 1902, provided with the platinum wire mentioned above. Lower rod vertically into the water as far as the wire can be seen; read the turbidity on the graduated rod at surface. Make observations in open air in quiet water, under natural conditions and in the shade.

Coefficient of fineness $= \dfrac{\text{suspended matter}}{\text{turbidity}}$ and is a measure of the size of the particles producing the turbidity.

Color is determined by comparison with standard platinum-cobalt solution. True color is that due to substances in true or in colloidal solution; apparent

* See page 773.

color includes that produced by suspended matter. Dissolve 1.246 grm. K_2PtCl_6 (0.5 grm. Pt) and 1 grm. $CoCl_2, 6H_2O$ (0.25 grm. Co) in water with 100 c.c. conc. HCl. Make up to 1 liter. This solution has a color value of 500. Dilute and compare sample in tubes by reflected light.

Odor. Observe odor both at 20° C. and at 90° C. Symbols used to describe odors are as follows:

v—vegetable	m—moldy
a—aromatic	M—musty
g—grassy	d—disagreeable
f—fishy	p—peaty
e—earthy	s—sweetish

Express degree of odor by the following numerals:

Numerical value	Term	Approximate definition
0	None	No odor perceptible.
1	Very faint	An odor that would not be detected ordinarily by the average consumer, but that could be detected in the laboratory by an experienced observer.
2	Faint	An odor that the consumer might detect if his attention were called to it, but that would not attract attention otherwise.
3	Distinct	An odor that would be detected readily and that might cause the water to be regarded with disfavor.
4	Decided	An odor that would force itself upon the attention and that might make the water unpalatable.
5	Very strong	An odor of such intensity that the water would be absolutely unfit to drink. (A term to be used only in extreme cases.)

Oxygen Consumed. $KMnO_4$, 0.4 grm. per liter; 1 c.c. = 0.1 mg. O. $(NH_4)\overline{Ox}$,* 0.888 grm. per liter: 1 c.c. = 0.1 mg. O. To 100 c.c. water (if much contaminated dilute smaller portion) in boiling flask add 10 c.c. C. P. H_2SO_4 (1 : 3), heat in water-bath 5 min.; add 10 c.c. $KMnO_4$ solution and heat 30 min. in water-bath; remove from bath; add 10 c.c. $(NH_4)\overline{Ox}$, then $KMnO_4$ till faint permanent pink. Each c.c. $KMnO_4$ used in excess is equivalent to 1 p.p.m. O.

Nitrogen as Free Ammonia. Standard NH_4Cl solution; 1 c.c. equals 0.01 mg. N. (0.0382 grm. NH_4Cl in 1 liter). Nessler's reagent: dissolve 50 grm. KI in water; add sat. $HgCl_2$ solution until a slight ppt. persists; add 400 c.c. of 50 per cent. KOH; settle and decant. Distil 500 c.c. of sample (with 0.5 grm. Na_2CO , if sample be acid); collect first three 50 c.c. portions of distillate in Nessler tubes; add 2 c.c. Nessler reagent to each tube; allow to stand 10 min. or more; compare with dilutions of standard NH_4Cl or with permanent standards. (See Standard Methods.†) NH_4Cl standard should be made up with ammonia-free water. Where free ammonia is very high it may be determined directly, first clarifying the sample by addition of 10 per cent. solu-

* Ox = Oxalic acid.
† Standard Methods of Water Analysis, American Public Health Ass'n.

tions of either $CuSO_4$, $Pb\overline{Ac}$* or $MgCl_2$, followed by sufficient 50 per cent. KOH to ppt. the metal as hydrate.

Nitrogen as Albuminoid Ammonia. Alkaline permanganate: 8 grm. $KMnO_4$ and 400 c.c. of 50 per cent. KOH to 1 liter. Boil off ammonia and make up to volume. Make blank determination and correct for ammonia remaining in reagent. After free ammonia is distilled from water, add 40 c.c. alkaline permanganate and distil until free of ammonia. Four or five 50 c.c. portions should be collected. Nesslerize and compare with standards.

Nitrogen as Nitrites. Sulphanilic acid solution: dissolve 8 grm. purest acid in 1000 c.c. 5 N.† acetic acid (sp. gr. 1.041).

A-amidonaphthalene solution: dissolve 5 grm. in 1000 c.c. 5 N. acetic acid; filter through absorbent cotton.

Standard nitrite solution ($NaNO_2$): dilute strong solution. (0.1 grm. nitrogen per liter) 1 part to 1000 parts of water. (1 c.c. = 0.0000001 grm. nitrogen.)

To 100 c.c. of sample in Nessler jars add 1 c.c. of each reagent; mix; allow to stand 15 min. and compare with samples similarly treated.

Nitrogen as Nitrates. Phenolsulphonic acid solution: heat 30 grm. phenol and 201 c.c. (370 grm.) C. P. H_2SO_4 for 6 hrs. in water-bath. Standard KNO_3; 7.2 mg. per liter, 1 c.c. = 0.001 mg. N. Prepare by evaporating 10 c.c. of strong solution (0.72 grm. KNO_3 per liter) to dryness; treat with reagent and make up to 1 liter. Evaporate 20 c.c. or less of the decolorized sample almost to dryness on the water-bath; add 1 c.c. of acid to residue; mix well; add water and either KOH or ammonia until alkaline. Compare with standards similarly treated. If chlorides interfere ppt. with Ag_2SO_4 and filter.

Nitrates may be determined by reduction, using 25 per cent. KOH and strip of aluminum foil weighing about 0.5 grm. Concentrate 100 c.c. of sample and 2 c.c. of KOH solution to 20 c.c. Pour into reduction tube and add aluminum foil. Close tube by means of rubber stopper provided with trapped outlet tube. After action is complete, distil off ammonia and nesslerize.

Hardness. Standard soap solution: dissolve 100 grm. dry white Castile soap in 1 liter 80 per cent. alcohol and allow to stand several days. From this solution take such quantity that the resulting solution made by diluting stock solution with 70 per cent. alcohol will give a permanent (5-min.) lather when 6.40 c.c. of it are properly added to 20 c.c. of standard $CaCl_2$ (use 75–100 c.c. stock solution).

Standard $CaCl_2$: dissolve 0.2 grm. pure $CaCO_3$ (calcite) in a little dilute HCl then add H_2O, and evaporate to dryness several times to expel acid. Make up to 1 liter with H_2O. 1 c.c. is equivalent to 0.2 mg. $CaCO_3$.

Put 50 c.c. of sample in a 250 c.c. bottle; add standard soap solution slowly until complete lather persists over surface of water 5 min. after shaking. If water be hard, take smaller portions and dilute to 50 c.c. with distilled H_2O. Estimate hardness by means of following table:

* $\overline{Ac}$ = acetic acid

† N, as adjective, = normal strength; 5 N = 5 times normal strength; $\frac{N}{50} = \frac{1}{50}$ normal strength

Table 307. Hardness: Parts per Million of Calcium Carbonate ($CaCO_3$) for Each Tenth of a Cubic Centimeter of Soap Solution When 50 c.c. of the Sample are Used

C.c. of soap solution	0.0 c.c.	0.1 c.c.	0.2 c.c.	0.3 c.c.	0.4 c.c.	0.5 c.c.	0.6 c.c.	0.7 c.c.	0.8 c.c.	0.9 c.c.
0.0.......	. .	... ..	. .	.. ...	. . .	. .	.. .	0.0	1.6	3.2
1.0.......	4.8	6.3	7.9	9.5	11.1	12.7	14.3	15.6	16.9	18.2
2.0.	19.5	20.8	22.1	23.4	24.7	26.0	27.3	28.6	29.9	31.2
3.0.......	32.5	33.8	35.1	36.4	37.7	39.0	40.3	41.6	42.9	44.3
4.0.......	45.7	47.1	48.6	50.0	51.4	52.9	54.3	55.7	57.1	58.6
5.0.	60.0	61.4	62.9	64.3	65.7	67.1	68.6	70.0	71.4	72.9
6.0.......	74.3	75.7	77.1	78.6	80.0	81.4	82.9	84.3	85.7	87.1
7.0.......	88.6	90.0	91.4	92.9	94.3	95.7	97.1	98.6	100.0	101.5

Table 308. Conversion Table of Hardness

	Parts per million	Clark degrees	French degrees	German degrees
Parts per million	1.0	0.07	0.10	0.056
Clark degrees.	14.3	1.00	1.43	0.80
French degrees.	10.0	0.70	1.00	0.56
German degrees	17.8	1.24	1.78	1.00

Hardness is expressed according to several arbitrary scales. Other methods for testing hardness are described in connection with water softening (pp. 720, 721).

Chlorine. Standard $AgNO_3$ solution, 2.40 grm. to 1 liter,—1 c.c. equivalent to 0.5 mg. Cl, as chlorides in the water.

Standard NaCl solution, 16.48 grm. per liter. 1 c.c. equivalent to 1 mg. Cl.

K_2CrO_4 indicator, 50 grm. per liter. Dissolve salt in a little water; add enough $AgNO_3$ solution to produce precipitate; filter, and make up to 1 liter with distilled water.

$Al_2(OH)_6$ for decolorizing: prepare by electrolyzing ammonia-free water using Al electrodes, or by precipitating 125 grm. alum, dissolved in 1 liter of water, with ammonia; wash precipitate formed until free from Cl, NH_3 and NO_2.

To 50 c.c. of sample in white porcelain evaporating dish, add standard $AgNO_3$ solution, using 2 c.c. of K_2CrO_4 as indicator, until red tint of Ag_2CrO_4 appears. If chlorine be high, use less than 50 c.c. and dilute to volume; if low, evaporate 250 c.c. or more to volume or add a known quantity of standard NaCl solution and titrate as directed. If color be above 30, add $Al_2(OH)_6$, heat and filter; if acid, neutralize with soda.

Iron may occur in water both in dissolved and suspended form in ferrous (unoxidized) and ferric (oxidized) condition. Total iron is determined as follows: Standard iron solution: 0.7 grm. $Fe(NH_4)_2(SO_4)_2$, $6H_2O$ in 50 c.c. H_2O, add 20 c.c. dilute H_2SO_4, warm slightly, oxidize with $KMnO_4$ and dilute to 1 liter; 1 c.c. is equivalent to 0.1 mg. Fe.

KSCN solution, 20 grm. to 1 liter. Conc. HC free from iron. Evaporate 100 c.c. of water to dryness; carefully ignite to destroy excess of organic matter; cool, add 5 c.c. conc. HCl; warm, add 10 c.c. H_2O and warm again. Transfer to 100 c.c. Nessler tube; filter if necessary and add a drop or two of sat. $KMnO_4$ solution. To cool solution add 10 c.c. of KSCN solution and make up to 100 c.c. To a blank tube containing reagent add standard iron solution until color matches that of sample. Compute amount of iron present from number of c.c of standard solution used.

For determination of iron in different degrees of oxidation, use Standard Methods.

Samples containing small quantities of iron should be concentrated. Samples containing large quantities of iron should be treated with HNO_3, and if necessary, $KMnO_4$; then the iron should be precipitated with ammonia, collected on a filter, redissolved and determined as above.

Manganese. Standard $KMnO_4$: dissolve 0.288 grm. $KMnO_4$ in 1 liter H_2O. 1 c.c. equivalent to 0.1 mg. Mn. HNO_3 (sp. gr. 1.1135) free from oxides of nitrogen. Sodium bismuthate, purest obtainable. Dissolve residue in 50 c.c. dilute HNO_3, oxidize twice by adding 0.5 grm. sodium bismuthate, heating after first oxidization until pink color disappears; filter through Gooch crucible, wash with 5 per cent. HNO_3 and compare with standards containing known amounts of standard $KMnO_4$. The $KMnO_4$ used for oxygen consumed method contains 0.139 grm. Mn per liter. Samples containing more than 10 mg. Mn per liter should be tested by Knorres' persulfate method. (See Standard Methods.)

Residue on Evaporation. Weigh platinum dish; evaporate to dryness 100 c.c. or more of sample; dry at 104° C. and weigh. Gain in weight is total residue in volume of water tested. Same procedure with filtered samples determines dissolved residue. Difference in weights of residues before and after ignition gives "loss on ignition." Final weight after ignition is that of "fixed residue."

Acidity. Titrate 100 c.c. of sample with $\frac{N^*}{50}$ Na_2CO_3, using erythrosine or methyl red. Methyl orange is unsuitable in presence of free aluminum sulfate. Express results in terms of $CaCO_3$.

Carbonic Acid. Standard $\frac{N^*}{22}$ Na_2CO_3. 1 c.c. = 1 mg. CO_2. Dissolve 2.41 grm. Na_2CO_3 in 1 liter CO_2 free water. Preserve in resistant glass bottles. Titrate rapidly 100 c.c. of sample, using phenolphthalein as an indicator. Use a Nessler tube and stir gently. Each c.c. $\frac{N}{22}$ $Na_2CO_3 \times 10 = 1$ p.p.m. free CO_2. When water is acid to phenolphthalein:

Bicarbonate	= 1.22 times the alkalinity.
Carbonic acid as bicarbonate	= 0.88 times the alkalinity.
Half-bound carbonic acid	= 0.44 times the alkalinity.
Carbonate	= 1.22 times the alkalinity with phenolphthalein.

* See footnote, p. 790.

Table 309. Relation between Alkalinity by Phenolphthalein and that by Erythrosine in Presence of Bicarbonates, Carbonates and Hydrates

	Bicarbonates	Carbonates	Hydrates
P=0	E	0	0
P<½E	E−2P	2P	0
P=½E	0	2P	0
P>½E	0	2(E−P)	2P−E
P=E	0	0	E

E = Erythrosine alkalinity. P = Phenolphthalein alkalinity.

Carbonic acid (CO_2) as bicarbonate = 0.88 times the alkalinity with erythrosine or methyl red.

Bound carbonic acid, as CO_2 = 0.44 times the alkalinity with erythrosine or methyl red.

$$\text{Half-bound carbonic acid} = \frac{\text{bicarbonate carbonic acid}}{2}$$

$$\text{Bound carbonic acid, as } CO_2 = \frac{1}{2}(\text{carbonic acid as carbonate})$$

Dissolved Oxygen—Winkler's method.

$MnSO_4$, 48 grm. to 100 c.c. water.

KI + NaOH, 36 grm. NaOH and 10 grm. KI to 100 c.c. water.

Dilute H_2SO_4, 1 : 1, sp. gr. 1.40.

Thiosulfate solution, 6.2 grm. $Na_2S_2O_3$ to 1000 c.c. water. 1 c.c. solution = 0.2 mg. O (or 0.1395 c.c. at 0° C. and 760 mm. pressure).

Starch — 1 per cent. solution sterilized.

Collect sample in 250 c.c. calibrated, stoppered bottles without exposure to or absorption of atmospheric air. Take temperature. Add by pipettes reaching below surface, 2 c.c. each of $MnSO_4$ and KI + NaOH solutions, respectively. Stopper bottle and allow ppt. to settle. Remove stopper, add H_2SO_4; replace stopper and dissolve ppt. by shaking. Remove contents to flask and titrate with thiosulfate using starch as an indicator.

$$\text{Oxygen, p.p.m.} = \frac{200N}{V}$$

$$\text{Oxygen, c.c. per liter} = \frac{139.5N}{V}$$

$$\text{Oxygen, per cent. saturation} = \frac{20{,}000N}{VO}$$

Where N = c.c. thiosulfate solution.
V = capacity of bottle in c.c. less volume of reagents (4 c.c.).
O = p.p.m. of O in water saturated at same temperature and pressure.

Free chlorine in waters, treated in excess, may be detected by KI and starch solution in the sample acidified with H_2SO_4. For quantitative estimation, ortho-toluidine method is used. (Ellms and Hauser, Jour. Ind. & Eng. Chem., Vol. 5, No. 11, 1913.)

Ortho-toluidine solution: Dissolve 1 grm. in 1 liter water containing 10 c.c.

conc. HCl. Add 1 c.c. of test solution to 100 c.c. of water and compare color with that produced in standard solutions containing known amounts of chlorine. Standardize chlorine solution against $\frac{N}{100}$ thiosulfate solution. Use redistilled water for diluting standards and compare rapidly. Permanent standards of $CuSO_4 + K_2Cr_2O_7$ may be used. Test will detect 0.005 p.p.m. free Cl.

Table 310. Solubility of Oxygen in Fresh Water and in Sea Water of Stated Degrees of Salinity at Various Temperatures When Exposed to an Atmosphere Containing 20.9 Per Cent. of Oxygen and Under a Pressure of 760 mm.*

(Calculated by G. C. Whipple and M. C. Whipple from Measurements of C. J. Fox)

Temperature, centigrade	Chlorine in sea water (parts per million)					Difference per 100 parts of chlorine per million
	0	5000	10,000	15,000	20,000	
			Milligrams per liter			
0	14.62	13.79	12.97	12.14	11.32	0.0165
1	14.23	13.41	12.61	11.82	11.03	0.0160
2	13.84	13.05	12.28	11.52	10.76	0.0154
3	13.48	12.72	11.98	11.24	10.50	0.0149
4	13.13	12.41	11.69	10.97	10.25	0.0144
5	12.80	12.09	11.39	10.70	10.01	0.0140
6	12.48	11.79	11.12	10.45	9.78	0.0135
7	12.17	11.51	10.85	10.21	9.57	0.0130
8	11.87	11.24	10.61	9.98	9.36	0.0125
9	11.59	10.97	10.36	9.76	9.17	0.0121
10	11.33	10.73	10.13	9.55	8.98	0.0118
11	11.08	10.49	9.92	9.35	8.80	0.0114
12	10.83	10.28	9.72	9.17	8.62	0.0110
13	10.60	10.05	9.52	8.98	8.46	0.0107
14	10.37	9.85	9.32	8.80	8.30	0.0104
15	10.15	9.65	9.14	8.63	8.14	0.0100
16	9.95	9.46	8.96	8.47	7.99	0.0098
17	9.74	9.26	8.78	8.30	7.84	0.0095
18	9.54	9.07	8.62	8.15	7.70	0.0092
19	9.35	8.89	8.45	8.00	7.56	0.0089
20	9.17	8.73	8.30	7.86	7.42	0.0088
21	8.99	8.57	8.14	7.71	7.28	0.0086
22	8.83	8.42	7.99	7.57	7.14	0.0085
23	8.68	8.27	7.85	7.43	7.00	0.0083
24	8.53	8.12	7.71	7.30	6.87	0.0083
25	8.38	7.96	7.56	7.15	6.74	0.0082
26	8.22	7.81	7.42	7.02	6.61	0.0080
27	8.07	7.67	7.28	6.88	6.49	0.0079
28	7.92	7.53	7.14	6.75	6.37	0.0078
29	7.77	7.39	7.00	6.62	6.25	0.0076
30	7.63	7.25	6.86	6.49	6.13	0.0075

* Under any other barometric pressure, *B*, the solubility can be obtained from the corresponding value in the table by the formula:

$$S' = S\frac{B}{760} = S\frac{B'}{29.92}$$

S' = solubility at B mm. = B' in.
S = solubility at 760 mm. = 29.92 in

Bacteriological examination for certain uni-cellular forms is important. Certain of these bacteria are normal inhabitants of all surface waters and some ground waters. Some bacteria are entirely harmless, some very harmful, and some indicative of possible presence of harmful forms. Each species has both its natural and its accidental habitat. Any examination of water does not reveal all the bacteria present; methods for this have not been discovered, but the number growing under standard conditions is an index of the amount of pollution or contamination and sometimes of the quantity of food material available for their growth. Under normal conditions a well water should contain less than a surface water, and surface water with sewage contamination should show increased numbers. Fluctuations in hights of streams and general water conditions have a marked effect upon numbers of bacteria in surface waters. Of far greater importance, however, than mere numbers is the detection of certain organisms known to accompany fecal pollution. Many normal water bacteria are unable to grow at 37° C., while almost all fecal organisms grow at this temperature. Unfortunately many harmless soil bacteria are able to grow at 37° C. (98° F.).

According to Savage (Elements of Water Bacteriology, 1906):

Water only at 37° C. (98° F.) showed 3 bacteria per c.c.
Water only at 21° C. (70° F.) showed 76 bacteria per c.c.
Water + soil at 37° C. showed 1630 bacteria per c.c.
Water + soil at 21° C. showed 1970 bacteria per c.c.

In 1903, Boston sewage tested at M. I. T.* Sanitary Research Station showed 3,660,000 bacteria per c.c. when grown on gelatin at 20° C.; 2,310,000 per c.c. when grown on agar at 37° C. In the latter case, 550,000 were acid formers.

Bacillus coli is one of the normal types of bacteria of the intestinal tracts of man and higher animals. Its presence usually is the result of human or animal pollution. Houston (London, 1902–1903) found 100 to 1,000,000 B. coli per gram in feces of seventeen normal persons. To make presumptive test for B. coli, inoculate fermentation tubes containing dextrose broth and lactose bile, respectively, with varying volumes of water, 0.1 to 10 c.c., incubate for 24 to 48 hrs. at 37° C. and observe formation of gas. With production of 10 per cent. or more of gas, B. coli is presumed to be present. Lactose-litmus-agar plates inoculated with gas-forming cultures (see p. 796) may be used to isolate B. coli which form acid, red colonies.

There are several species in the B. coli group, all characterized by fermentation of dextrose and lactose with gas production. These bacteria are short bacilli with rounded ends, non-spore-forming, facultative anaerobes, give positive tests with esculin, grow at both 20° C. and 37° C., do not liquefy gelatin for 14 days and are not stained by Gram's method. The group consists of four species—B. communior, B. communi, B. aerogenes, B. acidi-lactici. The first two species are separated from the second two by their gas production with dulcite, and the first of each of these two groups may be separated from the second by its gas production with saccharose.

* Mass. Institute of Technology.

STANDARD MEDIA AND OTHER PREPARATIONS

Nutrient Gelatin and Agar

Nutrient gelatin and agar shall be prepared as follows:

<table>
<tr><th></th><th>Gelatin</th><th>Agar</th></tr>
<tr><td>1.</td><td></td><td>Boil 10 or 15 grm. thread agar in 500 c.c. water for half hour and make up weight to 500 grm. or digest for 15 min. in autoclave. Let this cool to about 60° C.</td></tr>
<tr><td>2.</td><td>Infuse 500 grm. lean meat 24 hrs. with 1000 c.c. of distilled water in refrigerator.</td><td>Infuse 500 grm. lean meat 24 hrs. with 500 c.c. distilled water in refrigerator.</td></tr>
<tr><td>3.</td><td colspan="2">Make up any loss by evaporation.</td></tr>
<tr><td>4.</td><td colspan="2">Strain infusion through cotton flannel.</td></tr>
<tr><td>5.</td><td colspan="2">Weigh filtered infusion.</td></tr>
<tr><td>6.</td><td>Add 1 per cent. Witte's peptone, and 10 per cent. gold label sheet gelatin on dry basis.</td><td>Add 2 per cent. Witte's peptone.</td></tr>
<tr><td>7.</td><td colspan="2">Warm on water-bath, stirring till peptone and gelatin are dissolved and not allowing temperature to rise above 60° C. (140° F.).</td></tr>
<tr><td>8.</td><td></td><td>To 500 grm. of meat infusion add 500 c.c. of 3 per cent. agar, keeping temperature below 60° C.</td></tr>
<tr><td>9.</td><td colspan="2">Titrate, after boiling 1 min. to expel carbonic acid.</td></tr>
<tr><td>10.</td><td colspan="2">Adjust reaction to +1.0 per cent. by adding normal hydrochloric acid or sodium hydrate as required.</td></tr>
<tr><td>11.</td><td colspan="2">Heat over boiling water (or steam) bath for 40 min.</td></tr>
<tr><td>12.</td><td colspan="2">Restore loss by evaporation.</td></tr>
<tr><td>13.</td><td colspan="2">Readjust to +1.0 per cent. if necessary and boil 5 min. over free flame, constantly stirring.</td></tr>
<tr><td>14.</td><td colspan="2">Make up loss by evaporation.</td></tr>
<tr><td>15.</td><td colspan="2">Filter through absorbent cotton and cotton flannel, passing the filtrate through the filter until clear.</td></tr>
<tr><td>16.</td><td colspan="2">Titrate and record the final reaction.</td></tr>
<tr><td>17.</td><td colspan="2">Tube, using 10 c.c. of medium in each tube.</td></tr>
<tr><td>18.</td><td colspan="2">Sterilize 15 min. in the autoclave at 120° C., or for 30 min. in streaming steam on 3 successive days. Put the gelatin at once into ice-water till solidified.</td></tr>
<tr><td>19.</td><td colspan="2">Store in the ice-chest in a moist atmosphere, to prevent evaporation.</td></tr>
</table>

Lactose litmus agar shall be prepared in same manner as nutrient agar, with addition of 1 per cent. of lactose to medium just before sterilization. Reaction shall be made neutral to phenolphthalein. If medium is to be used in tubes, sterilized azolitmin solution shall not be added until just before final sterilization. If medium is to be used in Petri dishes, sterilized azolitmin solution shall not be added to medium until it is ready to be poured into dishes. More colonies and better general results are obtained, when litmus and lactose are sterilized separately and added to the plate with the neutral

agar at time of planting. Good results can, however, be obtained, if agar and lactose are mixed and sterilized in autoclave at 120° C. for 15 min. only. Azolitmin on the market varies much, much being entirely unreliable. A 1 per cent. solution of Kahlbaum's azolitmin, if boiled 5 min., readily dissolves and needs no correction for acidity if added to standard agar. Both total and red colonies may be counted after from 18 to 24 hrs. when incubated at 37° C. Such tests are sometimes used in control of filtration plants.

Lactose Litmus. Prepare in same manner as agar and add 10 grm. lactose per liter with peptone and 100 c.c. of 1 per cent. azolitmin solution before tubing and sterilizing.

Sugar Broth. To 1 liter of distilled water, add 3 grm. beef extract (Liebig's), 10 grm. peptone (Witte's), 10 grm. sugar (dextrose, lactose, etc.) and boil for 5 min. Make up loss by evaporation and determine reaction as in case of gelatin; neutralize for fermentation tubes and sterilize.

Lactose Bile. To fresh ox bile, add 1 per cent. lactose and 1 per cent. peptone; filter, tube and sterilize. Dried bile may be used.

Bacteria which ferment dextrose also ferment lactose liver broth. Both lactose bile and liver broth develop growths of degenerate bacteria better than sugar broths. Lactose bile has been found to exert a mild inhibitive effect on B. coli, and degenerate forms may not develop in this medium, which, however, prevents overgrowths of other bacteria, notably sewage streptococcus.

Other Media. Directions for making other media mentioned above, also milk, nitrate broth, peptone solution for indol test, esculin, etc., may be found in Standard Methods.

Species Work. Determination of the bacterial flora in water or sewage is not usually worth the labor involved, and in most cases gives no better idea of sanitary condition than tests for B. coli. For methods and classification, see "Microörganisms in Water," Frankland; "Bacteriological Examination of Water," Horrocks; "General Bacteriology," Jordan; "Water Supplies," Savage; "Elements of Water Bacteriology," Prescott and Winslow; "Untersuchung des Wassers," Ohlmüller-Spitta, and other treatises.

Fungi and Molds. Fungi are flowerless plants without chlorophyl or starch. They are either saprophytic or parasitic. Some authorities class bacteria among fungi, thus relating them to mushrooms. Fungi consist of a vegetable portion called mycelium, which forms the ground-work of common mold or mildew; and the fruit, which is borne at the ends of the branching filaments, consists of spore cells, sometimes highly colored. Well-known molds are Mucor, Penicillium and Aspergillus. In investigations of water supplies the most important fungi are Crenothrix and other so-called iron bacteria, which grow abundantly in waters containing iron and manganese. These organisms have the power of precipitating iron upon the exterior of their filaments, forming a sheath of iron oxide. Sometimes the growth of these fungi and the precipitation of iron is so great as to completely clog services and small mains. Sometimes it is such a nuisance that either abandonment or deferrization of the supply is necessary. Dying iron bacteria decompose and give off disagreeable odors and may impart a bad taste to the water.

See "Microscopy of Drinking Water," Whipple; "Manual of Bacteriology," Muir and Richie; "The Structure and Functions of Bacteria," Fischer; "Molds and Yeasts," Conn; "Die Eisenbakterien," Molish.

Microörganisms, according to Whipple, include all organisms which are invisible or partly visible to the eye, including the bacteria mentioned elsewhere. Microscopic organisms, or plankton, a subdivision of the broader class, do not require special culture media and may be examined with an ordinary microscope using 1-in. ocular and ½-in. objective. Detailed methods for identifying and enumerating organisms are given by Whipple, "Microscopy of Drinking Water." In brief they consist in concentrating 500 c.c. or less of the water by filtration through sand; suspension of the sand in 5 c.c. of water, 1 c.c. of which is placed in a special cell, 20 mm. × 50 mm. × 1 mm., and examined using the optical combination mentioned, also an eyepiece micrometer so graduated that it covers 1 sq. micron of the cell on the stage of the microscope. For field work Bunker has devised a sling filter and a simpler circular cell. Usually 10 or 20 squares are counted. Masses of zooglea and amorphous material, also the organisms themselves, are counted as standard units by most observers. One standard unit = 400 sq. microns; 1 micron = 0.001 mm. The sand used is washed and screened between 60- and 100-mesh sieves. If possible, observations should be made immediately after collection, as many microscopic organisms are so fragile that they disintegrate rapidly.

Plankton is determined by filtering a large volume of water in a Plankton net or filter, weighing or measuring the residue. Results are expressed as milligrams per liter. Plankton includes all suspended organisms.

INTERPRETATION OF RESULTS

An analytical examination of water consists of a series of tests and experiments, usually from 10 to 25 in number, to assist in ascertaining its past history and its present condition. For the former purpose, chemical examination is of the most value; for the latter, bacteriological; the value of these is increased many fold if accompanied by a careful sanitary inspection of the source of the sample. The important object of a water analysis is the determination of the presence or absence of unpurified human, animal or industrial wastes, particularly sewage. Therefore, in the language of the late Professor Kinnicutt, "It is very obvious that no single set of chemical standards will be safe to use in commending or condemning a given water." The opinion of the analyst must be based upon the analytical results considered in the light of the geological and sanitary conditions revealed by the inspection of the sources.

The following synopsis explains in brief the significance of the various tests. For a more technical explanation, refer to standard works on water analysis, such as:

American Public Health Ass'n, Report of Committee on Standard Methods of Water Analysis, 1913; Woodman and Norton, Air and Food, 1915; J. C. Thresh, The Examination of Water and Water Supplies, 1904; Ohlmüller and

Spitta, Untersuchung des Wassers, 1910; Leffmann and Beam, Examination of Water, 5th ed., 1915; G. C. Whipple, Microscopy of Drinking Water, 1914; Prescott and Winslow, Elements of Water Bacteriology, 1913; Kossowicz, Einfuhrung in die Mykologie der Gebrauchs und Abwasser, 1913.

Color is a measure of colored substances in solution, such as vegetable matter, dissolved from roots, leaves and swamps, also humus and iron salts, expressed in terms of Std. Pt.-Co. solution.

Odor is a measure of the odor produced by the various substances contained in the water, whether vegetable matter in solution, microscopic organisms or gases of decomposition. Odor is useful in detecting the presence of sewage. The microscopical examination is invaluable for determining causes of odors in surface water. Many of the causes of odors are obscure and need further study.

Turbidity is a measure of suspended matter which obstructs the passage of light. Turbidity may be due to silt, clay, suspended Fe, organic matter, microörganisms, etc. It is expressed in terms of the turbidity produced by a given weight of silica.

Oxygen consumed is a measure of the carbonaceous organic matter which is partly oxidized by acid $KMnO_4$ solution. Waters which oxidize rapidly usually contain unstable carbonaceous matter. Oxygen consumed is closely related to color.

Nitrogen exists in water in several states of combination. Organic nitrogen in oxidizing to nitrates, passes through an intermediate stage, nitrites, or may be decomposed with evolution of NH_3 or free N. "A state of change is a state of danger" (Drown).

N as Albuminoid Ammonia is an approximate measure of the nitrogenous organic matter. The method determines but part (about one-half) of the total organic nitrogen. This is from two sources—vegetable and animal. Proteids and amino bodies from vegetable sources are much more stable than those from sewage and evolve N less rapidly when treated with alkaline $KMnO_4$ and distilled. Vegetable albuminoid ammonia is accompanied by color and may be due in part to presence of microscopic organisms, algæ, crenothrix, etc. These are in suspension; therefore the importance of determining suspended albuminoid ammonia. In surface waters, albuminoid ammonia should not exceed 0.3 p.p.m., although waters may safely contain more. Pure ground water contains very little albuminoid ammonia, and those containing 0.15 p.p.m. or more are usually condemned.

N as free ammonia is a result of decomposition of organic nitrogen and usually indicates roughly the amount of decomposed material. More than 0.15 p.p.m. must be regarded with suspicion. Free ammonia in deep wells is often of fossil origin and has little sanitary significance.

Nitrites are an indication that either oxidation of organic nitrogen or decomposition of nitrates is taking place. Its presence in any considerable amount in drinking water must always be regarded with suspicion, although deep well waters of great purity often contain nitrites.

Nitrates are a measure of completely mineralized nitrogen and when present in considerable amounts are indicative of past contamination.

Alkalinity is a measure of the carbonates and bicarbonates and is included in the hardness determination.

Total hardness by soap is a measure of the soap-consuming power of the water expressed in terms of $CaCO_3$. It is a rough measure of the Ca and Mg salts present and includes both *Alkalinity* and *Mineral Acid Hardness*, which

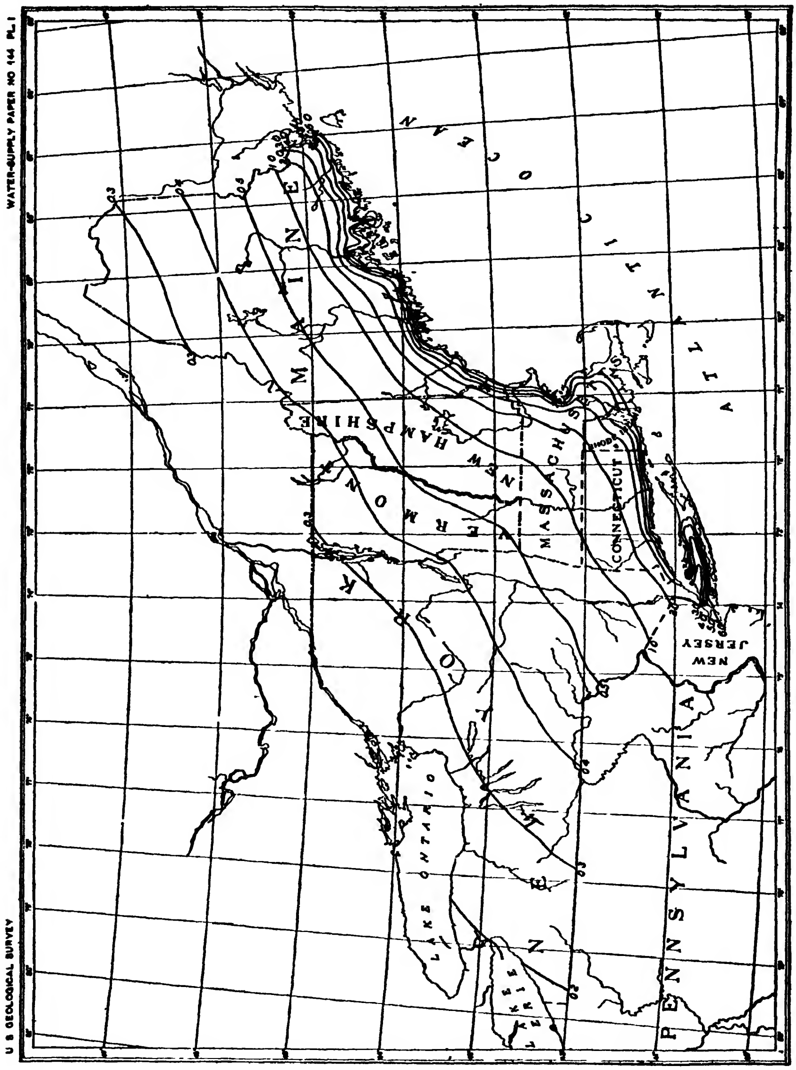

FIG. 411.—Normal Chlorine Map of New England and New York.

latter is due to presence of chlorides, sulphates and nitrates. Mineral acid hardness is often called *Permanent Hardness* or *Incrustants;* it is not removed by boiling.

Chlorine is found in all natural waters. Its source may be salt deposits in the soil. Chlorine is carried inland from the sea by the air currents, precipi-

tated with the rain. Where the normal chlorine is known, determination of chlorine is of very great sanitary significance. An excess, in absence of soil deposits, is a sure indication of present or past pollution. A map has been prepared for the Eastern portion of the United States. See Fig 411. In localities where salt deposits affect the water, determination of normal chlorine is impracticable.

Total residue is a measure of the matters dissolved and suspended in the water.

Poisonous Metals. Service pipes and metal vessels in contact with water often dissolve readily. This is usually due to the presence of carbon dioxide assisted by oxygen and by the absence of mineral constituents which would retard the action. Poisoning by copper, zinc and lead, most frequently by the latter metal, is likely to occur. A water containing as low as 0.5 p.p.m. of lead has caused lead poisoning, and even smaller amounts of any of these metals have been known to endanger health when taken regularly.

Bacteria are valuable indicators of the present condition of any water. The total number is of value in making routine tests from the same source, though liable to insignificant variations. Ordinary procedure enumerates only a part of the bacteria present. The proportion enumerated is variable. Of much more value is the determination of the members of the group of bacteria of which Bacillus coli is the type. Presence of these indicate pollution and possible presence of pathogenic bacteria, such as B. typhosus and B. paratyphosus. A search for B. enteriditis is often of value.

Examples. The following typical analyses illustrate far better than any set rules the meaning of analytical results.

Table 311. Comparative Analyses of Wholesome and Polluted Waters

(Parts per million)

Source of sample			Massachusetts mountain spring (unpolluted)	Unpolluted ground water; wells and filter gallery	(Metropolitan) Boston city supply; surface	Sewage effluent	Polluted well
Odor	Cold, 20° C		00 0	00 0	00 0	00 0	00 0
	Hot, 90° C		00 0	00 0	Vegetable	Musty	2 Vegetable
Turbidity—silica standard			1	....	....	9.	2.
Color—platinum standard			00.	20.	14	10	1.
Oxygen consumed			0 39	....	2 3	2.23	1.02
Nitrogen as	Free ammonia		0.000	0.045	0.013	0.430	0 092
	Albuminoid ammonia	Total	0.006	0.054	0.097	*5.64	0.074
		Suspended			0.015		
		Dissolved	....	....	0.072	....	....
	Nitrites		0.000	0.001	0.000	0.214	0 005
	Nitrates		0.000	0.246	0 031	8.87	36 0
Chlorine			3 00	7.2	2.8	42.2	142 5
Alkalinity			11 0	....	....	....	24 0
Hardness by soap method			14 3	48 0	11 0	....	117 0
Iron, Fe.			....	0 46	....	....	....
Total residue on evaporation			32 0	95 7	30 9	296 0	936 0
Bacillus coli in 10 c.c.			Negative	Negative	Negative	Positive	Positive
B. coli—presumptive test			Negative	Negative	Negative	Positive	Positive

* Total organic nitrogen.

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